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NATIONAL DAM INSPECTION PROGRAM. ROCK DAM (PA-00059), DELAWARE --ETC(U)
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DELAWARE RIVER BASIN
ROCK RUN CREEK, CHESTER COUNTY

PENNSYLVANIA
ID NO. PA.00059

ROCK RUN DAM

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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Contract No. DACW31-78-C-0048

DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

AUGUST 1978

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LEVEL II

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DELAWARE RIVER BASIN

ROCK RUN DAM
CHESTER COUNTY, PENNSYLVANIA
NATIONAL I.D. NO. PA 00059

6 National Dam Inspection Program. Rock Run Dam (PA-00059), Delaware River Basin, Rock Run Creek, Chester County, Pennsylvania. Phase I Inspection Report.

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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Prepared by:

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Submitted to:

DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

Name of Dam: Rock Run Dam
County Located: Chester County
State Located: Pennsylvania
Stream: Rock Run Creek
Coordinates: Latitude 40° 00.3' Longitude 75° 51.3'
Date of Inspection: 18 August 1978

Rock Run Dam is a hollow reinforced concrete structure of the "Ambursen" type approximately 583 feet long and 42 feet high. The buttress walls are founded in rock and there is an upstream concrete cutoff wall extending at least 8 feet into natural rock. The dam was built in 1917. The dam has been in service since construction and has experienced some concrete spalling and deterioration. The dam is assessed to be in fair condition.

Hydrologic and hydraulic computations presented in this report indicates that the dam will only pass 30 percent of the probable maximum flood (PMF) without overtopping. At this flow, overtopping would first occur adjacent to the abutments and then over the dam. Overtopping could cause failure of the structure starting with erosion adjacent to, and possibly under, the retaining walls at each end of the dam as these end walls were not founded in rock. Since the dam will overtop at flows less than 50 percent of the PMF, coupled with possible structural failure, the spillway is considered to be "Seriously Inadequate".

The dam is classified as an "Intermediate" size dam by virtue of its 42 foot height and 1200 acre-feet normal storage. The dam is also considered to be a "High" hazard structure because in the event of failure, there is a possibility of loss of life and extreme property damage downstream.

The following recommended remedial work should be undertaken immediately and is presented in order of priority. However, this does not infer that the latter recommendations are not important.

1. A concrete retaining wall should be installed at each end of the dam to an elevation of at least as high as the parapet walls. This will

enable the structure to pass higher flows and changes the classification of the spillway from "Seriously Inadequate" to "Inadequate" in that the spillway will then pass an estimated 68 percent of the PMF.

2. The pond drain system should be rehabilitated so the reservoir can be drained in the event of an emergency. Alternately, another drainage system should be installed.

3. The spillway and flood storage capacity of the reservoir should be evaluated by a registered professional engineer and redesigned to meet current state-of-the-art hydrologic/hydraulic standards.

The following recommendations are considered important and should be attended to within one year.

1. A registered professional engineer should evaluate the downstream struts between buttresses. As necessary, these struts should be rehabilitated or recoated with suitable materials to prevent further deterioration of the reinforcing steel.

2. Rehabilitation should also include the downstream plaster filler walls and repair of the spalled concrete along the spillway and the parapet wall.

The Owner should develop an inspection checklist together with an inspection and maintenance procedure to insure that all items are properly and periodically inspected, operated and maintained. Because of the downstream population, a formal procedure of observation and warning during periods of high precipitation should be developed and implemented. This procedure should include a method of warning downstream residents that high flows are to be expected along the creek.

John H. Frederick, Jr., P.E.
Maryland Registration 7301
Woodward-Clyde Consultants

Date 9/25/78

W S Gardner

William S. Gardner, P.E.
Pennsylvania Registration 4302E
Woodward-Clyde Consultants

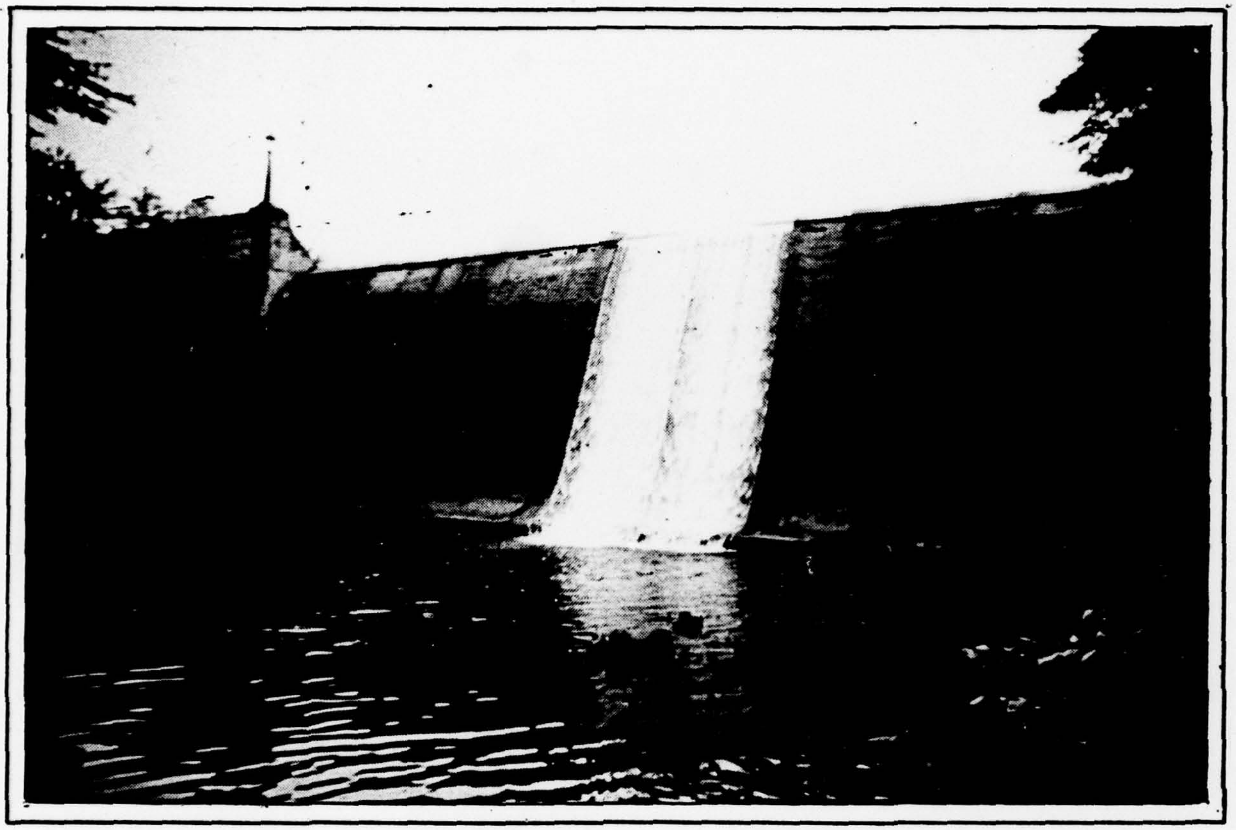
Date 9/25/78

APPROVED BY:

G. K. WITHERS
Colonel, Corps of Engineers
District Engineer

Date 28 Sep 78

Under the recently revised spillway evaluation guidelines, this dam is considered unsafe, non-emergency.



OVERVIEW
ROCK RUN DAM, CHESTER COUNTY, PENNSYLVANIA

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
ROCK RUN DAM
NATIONAL ID #PA 00059
DER #15-4

SECTION I
PROJECT INFORMATION

ABSTRACT

1.1 General.

a. Authority. The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

b. Purpose. The purpose of the inspection is to determine if the dam constitutes a hazard to human life or property.

ABSTRACT

1.2 Description of Project.

a. *Dam and Appurtenances.* Rock Run Dam is a hollow reinforced concrete structure of the "Ambursen" type, approximately 583 feet long and 42 feet high. The dam is founded on rock with an upstream concrete cutoff wall and grout curtain. The dam was built between the summer of 1915 and the spring of 1917.

As shown on Plate 2, Appendix E, the dam is a concrete slab and buttress structure with the upstream slab inclined at an angle of 40° from the horizontal. The downstream batter is 1H:4V. The dam contains 27 buttress walls formings 26 bays supported directly on decomposed or intact rock. Each bay is 15 feet on center for a total length of 513 feet. Wing walls extend beyond the buttress walls, increasing the length of the dam to approximately 583 feet. The spillway is located between bays 10 through 17 with a notched section between bays 13 and 14, as shown on Plate 2. The crest elevation is 485 feet (MSL) at the notch and the remainder of the spillway is 2 inches above the notch. The parapet walls extend to elevation 492.

The dam was designed for water supply and contains three intake pipes as shown on Plate 3. The two upper pipes are 18 inches in diameter with elevations at 473.0 and 463.1, respectively. The lower 24-inch cast iron pipe is at elevation 452.8 and serves as a pond drain. All three pipes feed into a common 24-inch cast iron pipe and pass through the downstream toe. At the present time the two lower pipes are inoperable; the valves are rusted closed and silt has accumulated to an elevation above 465. Currently, water is supplied by demand through the upper intake pipe and to the water treatment plant immediately downstream. A second pond drain pipe is located in Bay No. 11, but it is not known how and when this pipe

was used. It is believed possible that this pipe was used for diversion during some period of construction.

b. Location. Rock Run Dam is located on Rock Run in West Caln Township, Chester County, Pennsylvania. The dam is located approximately 1.6 miles above the confluence of Rock Run and the West Branch of Brandywine Creek.

The dam site and reservoir are shown on USGS Quandrangle entitled, "Wagontown, Pennsylvania", at coordinates N 40° 00.3' W 75° 51.2'. A regional location plan of Rock Run Dam is enclosed as Plate I, Appendix E.

c. Size Classification. The dam is classified as "Intermediate" by virtue of its 42 foot height and 1,200 acre-feet normal storage.

d. Hazard Classification. A "High" hazard classification has been assigned to this dam because of the possibility of extensive property damage and loss of life downstream in the event of failure of the structure.

e. Ownership. The dam is owned by the City of Coatesville. All correspondence should be sent to the attention of Mr. Don Thompson, City Administrator, City Hall, Water Department, 53 South First Avenue, Coatesville, Pennsylvania 19320.

f. Purpose of Dam. The purpose of this dam is to supply water to the town of Coatesville and the surrounding areas. Fishing is also permitted, but limited to people with a special permit.

g. Design and Construction History. Between 1909 and 1914, several water supply studies were made to evaluate the possibility of constructing the dam on one of several tributaries of Brandywine Creek. During these investigations, six test pits were excavated approximately 10 feet deep to intact rock along the present center line of the dam. Based on these test pits and other surficial investigations, it was concluded that this was the best site to found the structure. The application report was submitted in 1914. The report of the application was approved by Mr. J. E. Seelye for the State of Pennsylvania in February, 1914. Between 1914 and March 1915, several letters and design changes were made by the Consulting Engineer, Mr. Alexander Potter of New York City, New York.

Construction began in August of 1915, and by October, several buttress wall footings were excavated and approved. Water pressure tests were performed in mid-October, 1915, and core samples were taken. During this period, holes were drilled along the centerline of the dam 50 feet on center and approximately 20 feet deep. Rock cores indicated that the foundation rock was weathered and that grouting would probably be necessary. Mr. Potter recommended that 3-inch holes be drilled 5 feet on center and pressure grouted to seal the foundation. The actual grouting work was postponed until August 1916.

By 23 December 1915, the weather was too cold to place concrete without suitable protection. Therefore, the State directed/recommended that the

aggregate and water be heated before mixing the concrete to prevent freezing of the concrete. The contractor installed steam pipes which ran through the aggregate piles, and inspection reports confirmed that this system of heating the aggregate prior to mixing was acceptable. During foundation preparation, several photographs were taken, and these photographs indicated that all foundations were constructed on either intact or partially decomposed rock.

On May 23 and May 24, 1916, rainfall runoff breached the diversion dam, flooding the foundation areas. This delayed construction for approximately one week, until the area could be cleaned and reevaluated. By 12 June 1916, the State and other engineering representatives concluded that foundation grouting was necessary to prevent leakage under the dam. The State representatives recommended that grout holes be drilled 5 feet on center to a depth of at least 25 feet, and the split spacing technique would be used. They also recommended that grout pressures range between 40 and 50 psi. On 4 August 1916, Sprague and Henwood Company arrived at the site and began foundation grouting. Records indicate that their grouting was performed in accordance with the specifiations, and the grout pressures ranged from 0 to as much as 110 psi, averaging 40 psi. There was notable grout communication between holes, first in the form of clear water, then in a form of diluted grout and finally grout approximating the consistency of the grout being pumped. Foundation grouting was terminated when pressures exceeded 50 psi, or when the consistency of the grout leakage was the same as the grout being pumped.

By December 1916, most of the essential structural features of the dam were complete. Thereafter, mostly facial and cosmetic work was performed. It was then concluded that an inspector was no longer necessary, since the work that was being performed did not relate to the structural integrity of the dam. The last inspection report in the Department of Environmental Resources files was dated April, 1917. The dam was considered complete in the spring of 1917.

From the start of construction, in August 1915, through the fall of 1916, construction was very slow. The job was riddled with construction and labor problems, and slowed by several design changes and protests from residents. The State was very concerned about assuring that the foundation was securely seated into dense and relatively unweathered rock. There was also concern that differential settlement would occur causing structural cracking of the dam. Several assessments were made, including re-analysis of the structure and the stability against sliding and overturning. State personnel were also concerned with the fact that much of the concrete work was being poured in the winter time.

Many photographs were taken to document that each buttress wall was founded securely into rock. These concerns led to several arguments between the contractor, the City and the State. Subsequently, a reorganization of personnel occurred in the fall of 1916 and construction improved significantly.

In November 1916, the assistant engineer for the State of Pennsylvania requested that a 20-foot section of the spillway be redesigned to contain a 20-foot notch 2 inches deep at elevation 485. The purpose of this notch was to funnel

normal runoff water through a restricted section of the spillway. This notch was approved in a letter dated 15 November 1916 by the consulting engineer. In addition, on August 3, 1916, permission was given to relocate a 24-inch pipe, but there are very few details as to where this pipe was located. During the visual inspection a 24-inch pipe was noted in Bay No. 11 and it is assumed that this is the pipe that is described in correspondence dated August 1916.

Between the date of completion and 1948, there are very few records concerning this structure. In September 1949, an application was submitted to replace the concrete weir cap on the spillway. The redesign was performed by Albright and Friel of Philadelphia, Pennsylvania, and the work was performed by Edward A. Daylor, Construction Company, Inc. There are no other records concerning modifications or changes to this structure. Discussions with the Owner during the field inspection indicated that several water supply pipes were recently replaced between the dam and the new water treatment facility.

h. Normal Operating Procedure. Under normal operating conditions, the upper intake pipe, as shown on Plate 3, is left open and water is supplied by gravity to the treatment plant, approximately 400 feet downstream. It is reported by the Owner's representative that the lower and middle pipes are rusted and the valves are inoperable. In addition, the Owner believes that silt has accumulated to some elevation above the middle intake pipe. Excess water passes over the spillway into the downstream channel and subsequently into Brandywine Creek. There are no minimum downstream flow requirements.

1.3 Pertinent Data.

Pertinent data for Rock Run Dam is summarized as follows:

a.	Drainage Area (sq. miles)	5.3
b.	Discharge at Dam Site (cfs)	
	Maximum Design Flood	3,560
	Top of Dam	8,230
c.	Elevation (feet above MSL)	
	Spillway Crest (Notch)	485.0
	Spillway Crest (Main Crest)	485.2
	Top of Dam	489
	Top of Parapet Wall	492
	Intake Elevation	
	Top	473.0
	Middle (Blocked)	463.1
	Bottom (Blocked)	452.7

	Normal Pool	485.0
	Diversion Pipe	Unknown
d.	Reservoir (miles)	
	Length at Normal Pool	0.7
	Fetch at Normal Pool	0.5
e.	Storage (acre-feet)	
	Normal Pool	1,200
	Top of Dam	1,250
f.	Reservoir Surface (acres)	
	Normal Pool	61
g.	Dam Data	
	Type	"Ambursen" type hollow reinforced concrete
	Length	583 feet
	Maximum Height	42 feet
	Cutoff	Upstream concrete wall into rock.
	Grout Curtain	Yes. 3-in. grout holes, 5 feet o.c. at least 25 feet deep. Average grout pressure = 40 psi.
h.	Diversion	Initial diversion via a dam and temporary pipe. Later diversion through pond drain.
i.	Discharge	
	Water Supply	
	Type	2 cast iron pipes embedded in upstream face
	Sizes	18 inches
	Comment	Lower intake non-functioning.
	Spillway	
	Type	Concrete ogee weir with 20-foot long 2-inch deep notch.
	Length (Total)	117 feet
	Pond Drain	
	Type	Cast iron pipe embedded up stream face. Non-functioning.
	Size	24 inches

SECTION 2
ENGINEERING DATA

2.1 Design.

A summary of engineering data for Rock Run Dam is presented in the checklist attached as Appendix A. Principal documents containing pertinent data used for this report are as follows.

1. "Report Upon the Application of the Borough of Coatesville for the Construction of Rock Run Dam", Report No. 371, Chester County, Pennsylvania.

2. "Supplementary Report Upon the Application of the Borough of Coatesville", Permission to Construct the Dam on Rock Run, dated March 25, 1915.

3. "Supplementary Report Upon the Application of the Borough of Coatesville", for Permission to Construct a Dam on Rock Run, Report No. 15-4-4, dated 25 October 1915, prepared by George S. Beal, Assistant Engineer.

4. "Computations by Professor McKibben on the Borough of Coatesville Dam", dated 7 April 1914. This 13-page document contains a complete structural analysis of the "Ambursen" type hollow reinforced concrete dam.

5. "Report On the New Source of Water Supply from the Brandywine Creek for the Borough of Coatesville, Pennsylvania", prepared by Mr. T. Chalkley Hattan, Consulting Engineer, January 14, 1907.

6. "Report On Water Supply of Coatesville, Pennsylvania", by Myron L. Fuller, Consulting Engineer, dated August 8, 1913.

7. "Water Supply Investigation for the Borough of Coatesville, Pennsylvania", prepared by Sanderson and Porter, Engineers and Contractors, New York City, New York, dated August 9, 1913.

8. "Improved Water Supply for the Borough of Coatesville, Pennsylvania, Report Upon the Details of Construction of the Proposed Rock Run Dam And the Proposed High Service Distribution System", prepared by Alexander Potter, Consulting Engineer, New York City, dated December 30, 1913.

9. "Specification for Contract A, Rock Run Storage Dam and Reservoir", prepared by Alexander Potter, Consulting Engineer, New York City, dated 1913.

10. Assorted as-built sketches, stability calculations during construction, water supply and rainfall records during construction, miscellaneous notes, letters, and other assorted correspondence.

11. 137 black and white photographs covering preconstruction, construction, and post-construction documentation. In addition, there were many as-built drawings available in the files.

The data in DER files was quite comprehensive and covered most aspects of construction.

b. Design Features.

The principal design features of Rock Run Dam are illustrated on the plans, profiles, and cross-sections enclosed in Appendix E as Plates 2 through 4, and are described in Section 1.2, Paragraph a. These plates are reproduced from 1913 through 1916 drawings prepared by the City and the Design Engineer. Copies of the 1949 drawings prepared by Albright and Friel are also included in Appendix E.

2.2 Construction.

A description of the construction history is presented in Section 1.2. In summary, the original work was performed by the B. G. Coon Construction Company of Luzerne, Pennsylvania. The foundation grouting was done by Sprague and Henwood Company. Mr. Alexander Potter, Consulting Engineer of New York City, did construction consultation and design work. Mr. Myron L. Fuller, Consulting Engineer, directed the field investigation and also provided construction consultation. Professor McKibben performed structural computations prior to, during and after the construction period.

In 1949, Albright and Friel of Philadelphia, Pennsylvania, was hired to repair this structure. Their principal work was to prepare specifications for replacing the cap on the spillway and to provide recommendations for other minor repairs. The construction work was performed by the Edward A. Dayior Construction Company in 1949.

Since 1949, there has been no major reconstruction work on the dam. The upper water supply intake valve was reported to be replaced and a flow meter attached to the system. New pipes were constructed between the dam and the water treatment plant several years ago. Some refacing work was performed to the downstream slabs by Lansdowne Construction Company, but does not affect the structural components of the system.

2.3 Operation Data.

The only operational records maintained for this dam are reservoir levels and water supply flows. There is no operational manual. Under normal

conditions, water enters the water supply system through the upper intake pipe, and through an 18-inch pipe to the water treatment plant. It is understood from the operational personnel that the lower pipe and pond drain pipe are silted in and that the valves are rusted. During the inspection, these valves were turned and confirmed that they did not operate properly.

2.4 Evaluation.

a. Availability. Most engineering data reproduced in this report and studied for this investigation were provided by the Pennsylvania Department of Environmental Resources. Several drawings contained as Plates 2 through 4 were provided by the Coatesville City representatives.

b. Adequacy. The data available was very comprehensive and included a complete structural evaluation including stability analysis. There are several other documents describing results of the analysis and the consequences of design adjustments made. There was a complete set of daily, weekly and, in some cases, monthly construction progress reports. There were 137 photographs and foundation approval slips signed by State representatives, which verify that the foundations were founded on competent rock. There were several miscellaneous letters and other pieces of correspondence describing how construction was performed, and describing many of the problems associated with the construction. The visual inspection of the dam described in Section 3 confirms that the dam was constructed essentially as designed.

c. Validity. There is no reason to question the validity of the data. There were records in the files by various consultants who reviewed the structural analysis and other redesigned changes. All of these people verified that the redesign was correct and conservative.

SECTION 3
VISUAL INSPECTION

3.1 Findings.

a. General. The observations and comments of the field inspection team are contained in the checklist enclosed herein as Appendix B, and are summarized and evaluated as follows. In general, the appearance of the facility indicates that the dam is maintained and the structure is in good condition. An assessment of the water supply system indicates that the water supply pipes within the dam are in poor condition, as the lower two pipes do not work, and the upper pipe valve is, for all practical purposes, locked in the open position.

b. Dam. During the visual survey, there were no indications or evidence observed of distortions in alignment or in movement of the crest that would be indicative of foundation settlement or imminent failure of the structure. There were no signs of distress on the upstream face as viewed from inside the structure from the observation platform. Many struts between buttresses on the downstream face have had their bottom reinforcement exposed through weathering, and in some cases the reinforcing steel is rusted and deteriorating. Vertical reinforcing steel was occasionally exposed in the buttress walls. Often the steel was rusted and deteriorated. All of these exposed areas were along the downstream half of the structure.

The plaster filler walls on the downstream face between buttresses were often cracked, buckled, and generally in poor condition. It is noted, however, that these are not structural components, but only facial components. Although the reason for installation of the plaster filler walls was not found in these files, analysis of other similar dams indicates that the downstream facing was installed for two purposes. The first and primary purpose was to minimize temperature changes on the underside of the upstream slab, thus minimizing spalling and deterioration of the slab. As a secondary purpose, the downstream facial wall was placed for aesthetic reasons.

Construction joints on the underside of the upstream slab, showed signs of leaching, but there were no significant signs of deterioration. The principal structural elements, such as the buttress walls, were assessed to be in good condition. There were some localized spalled areas with reinforcing steel exposed and deteriorated. The downstream struts connecting buttresses were the only major structural elements which showed significant signs of deterioration. However, it is noted that no major structural cracks were observed to indicate possible instability of the structure.

Seepage was emanating through the foundation, but this seepage was clear and assessed to be relatively small in quantity. Due to the location of the seepage, and the insignificant quantity, an assessment of the rate of flow was not determined.

c. Appurtenant Structures.

1. Intake Pipes. The intake pipes are assessed to be in good condition, but the valves in the pipes are assessed to be in poor condition. It is noted that only the upper intake pipe is functioning, and it is reported by the plant operator that that valve is essentially locked in the open position. It is reported by the Owner's representative that the middle intake pipe, although observed to be in good condition, is silted-in and the valve is rusted shut.

The lower 24-inch pond drain pipe could not be inspected since it was covered with soil. Exposed portions of the gate valve were observed and it was leaking at the stem. An attempt was made to exercise this valve, but the leakage became excessive and further exercising of the system was terminated.

2. Spillway. The spillway was inspected and observed that the crest was in good condition. However, the spillway chute below the cap was observed to be spalled and some displacement was noted between the downstream chute and the weir cap. The stilling basin immediately below the spillway is rock lined and relatively stable. Water flows over a notched 25-foot diameter semi-circular weir, and then discharges into the downstream channel.

d. Reservoir. Reconnaissance of the reservoir disclosed no evidence of slope instability or other features that would significantly affect the flood storage capacity of the reservoir. All slopes were well vegetated with an assortment of hardwood and softwood trees, and the surrounding drainage basin was partially residential, partially wooded, and partially commercial/recreational (golf course).

It was reported by the Owner that the upper end of the reservoir was dredged about 1972. There was no dredging reported at the downstream section. The operating personnel who accompanied the inspection team indicated that there was probably significant quantities of silt on the upstream face of the dam, resulting in nonfunctioning middle and lower intakes.

e. Downstream Channel. Immediately downstream of the spillway, the discharge enters a stilling pool and flows over a 25-foot diameter semi-circular overflow weir. Thereafter, the water flows downstream along a gravel bottom channel for approximately 2.3 stream miles before entering the West Branch of Brandywine Creek. The drainage area along the stream is predominantly wooded or farmland. See Section 5 for additional discussion on downstream conditions.

3.2 Evaluation.

In summary, the visual survey of the dam disclosed that there was no significant structural cracking of the dam, and it is assessed that the dam itself is in fair condition. The downstream facial walls, on the other hand, are in poor

condition, in that they are cracked, buckled and reinforcing steel and wire mesh facing is exposed. Typical photographs taken during the visual inspection are shown in Appendix D. There was some minor seepage noted through construction joints and some hairline cracks. However, this seepage is not considered to be indicative of a potentially unstable condition.

The intake pipes were inspected and found to be in good condition. It is noted that the valves are in poor condition and that the lower two intakes are blocked with silt.

The overall assessment of the dam and appurtenances is that the structure is considered to be in fair condition with the understanding that the pond drain systems are currently nonfunctional. The reservoir cannot be drained by the methods designed into the structure.

SECTION 4
OPERATIONAL PROCEDURES

4.1 Procedures.

Normal operating procedure does not require a dam tender. Water is supplied to residents by an 18-inch intake at elevation 473.0. Water travels through another cast iron pipe directly into the treatment facility, and is thereafter distributed to the residents of Coatesville and the surrounding area. Excess water is discharged over the spillway and into Rock Run.

4.2 Maintenance of the Dam.

There are no formalized maintenance procedures for this dam. It appears that there was little or no maintenance performed to the water supply systems located within the dam, in that the valves were rusty and locked in their current position. Several years ago, selected areas of the upstream face were repaired and resurfaced with grout. Since that time, there have been no major repairs performed to the dam.

4.3 Maintenance of Operating Facilities.

There are no maintenance procedures delineating the requirements for maintaining the operating facilities. It appears that the water supply pipes/valves are not maintained in that they are always in the open position and water is fed by gravity to the treatment plant and distributed on demand.

4.4 Warning Systems In Effect.

There are no formal warning systems or procedures established to be followed during periods of exceedingly heavy rainfall. However, representatives at the water treatment facility monitor the dam during periods of exceedingly heavy rainfall. The Owner's representative also indicates that the external portions of the dam are inspected daily to determine if unusual seepage is developing downstream.

4.5 Evaluation.

There are no operating procedures nor are there any warning systems or procedures established to be followed during periods of exceedingly heavy rainfall, or in the event of an emergency. Commensurate with the possibility of loss of life and extreme property damage downstream in the town of Rock Run in the event of failure, a formal warning procedure should be implemented. An operating procedure, together with an inspection checklist should also be formulated and

implemented by the Owner. Coupled with this operational manual, a maintenance procedure should also be formulated. The listing of items to be inspected should include all critical items of the facility.

It is also assessed that the pond drain pipe should be reestablished so that it will operate in the event that it is necessary to draw down the reservoir. It is determined, based on discussions with the Owner's representative, that this pipe is blocked and will not operate.

During the inspection, several other pipes were found; one is located inside the dam in Bay No. 11 and two pipes were located on either side of the stilling basin pool. The Owner's representative did not know what these pipes did or where they went. It is recommended that these pipes be traced to determine their function and their possible use in draining the reservoir.

SECTION 5
HYDROLOGY/HYDRAULICS

5.1 Evaluation of Features.

a. Design Data. Available design data relating to the design of this dam was contained in the application report dated January 30, 1914. Although the designer, Alexander Potter, Consulting Engineer, submitted a spillway design "of ample size to discharge the greatest flood likely to occur during the life of the structure," Charles Ryder's application report(1) demonstrated a larger spillway was required. The drainage area was subdivided into 11 areas, and the peak discharge rate from a two-hour storm, 5.48 inch rainfall occurring on August 3, 1898, was determined. It was calculated that the peak inflow rate of 4,772 cfs could be safely discharged by a 117-foot long spillway. Spillway capacity is 2,310 cfs with a 3-foot head. The drainage area used in the calculations was 5.32 square miles, which does not agree with the total drainage area shown on the watershed map attached to the application, 6.67 square miles. The drainage area disclosed by current USGS maps is 5.69 square miles.

The watershed is currently 40 percent wooded, 10 to 15 percent residential, and the remaining is open farmland. Elevations range from 840 feet in the upper reaches to 485 feet at the normal pool elevation. It is expected that residential development will continue at a moderate rate within the watershed for the foreseeable future.

In accordance with the criteria established by the Federal (OCE) Guidelines, the recommended spillway design flood for this "Intermediate" size dam and "High" hazard potential classification is the probable maximum flood (PMF).

b. Experience Data. The plant maintains only water supply usage records and limited documentation on reservoir levels.

c. Visual Observations. On the date of the inspection, no conditions were observed that would indicate that the outlet capacity would be significantly reduced during a flood occurrence. Although siltation has occurred at the upper end of the reservoir, and reportedly there are significant quantities of silt at the lower end of the reservoir, the effect on flood water storage capacity is minimal. Observations regarding the condition of the downstream channel, spillway conditions, and reservoir, are located in Appendix B.

d. Overtopping Potential. The PMF peak inflow is estimated to be 13,200 cfs when compared to the nearby Marsh Creek Dam Watershed (see Appendix C).

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(1) See Section 2.1, reference No. 1.

Marsh Creek Dam was also inspected under the National Dam Inspection Program. Marsh Creek peak PMF inflow was developed according to the Soil Conservation Service criteria, which is conservative for small watersheds. The spillway capacity when the reservoir level is at the top of the dam is about 3,560 cfs. Flood routing through the reservoir by the approximate method (Sheets 9 and 10, Appendix C) indicates that the spillway capacity and flood storage are insufficient to pass either the PMF or 0.5 PMF without overtopping. At these storms, the water elevations would be 493.5 and 490.6, respectively. This corresponds to 8.5 and 5.6 feet above the spillway crest for the PMF and 0.5 PMF conditions.

Although the dam has core walls extending into each abutment, the drawings indicate that these do not extend horizontally into bedrock. Thus, it is possible that if the dam overtops, some erosion will occur along both sides of the core wall, which could lead to failure of the structure.

If the abutments are raised to the same elevation as the parapet wall, the spillway capacity would be approximately 8,230 cfs. The increased spillway capacity and the increased flood storage are sufficient to approximately 0.67 PMF without overtopping the dam.

e. Spillway Adequacy. The spillway system is judged "Seriously Inadequate" if all of the following conditions exist (Engineering Technical Letter No. 1110-2-234, 10 May 1978):

> "1. There is a high hazard to loss of life from large flows downstream of the dam.
>
> "2. Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just prior to overtopping.
>
> "3. The dam and spillway are not capable of passing one-half of the probable maximum flood without overtopping failure."

It is judged that Rock Run Dam would fail as a result of overtopping and erosion around the abutments. Drawings indicate that the core walls do not extend to rock and that failure would first occur at the ends of the dam. Without increasing the height of the dam, the dam is capable of discharging only 0.3 PMF without overtopping (see Appendix C for details). In the event that failure occurs, all of the aforementioned conditions would be satisfied and the spillway is considered "Seriously Inadequate".

As shown in Appendix C, the dam is capable of passing 0.68 PMF without overtopping when the abutments are raised to the same elevation as the parapet wall. The spillway would still be rated as "Inadequate", but not "Seriously Inadequate" in that it will pass more than 0.5 PMF without overtopping.

f. Downstream Conditions. Discharge from Rock Run flows under Waterworks Road approximately 250 feet downstream of the dam. It is expected that for flows greater than 5,000 cfs, the bridge would be flooded.

Within the first 1.5 miles below the dam, there are approximately 2 houses subject to damage if the dam fails. Approximately 2 miles downstream of the dam, Rock Run flows through the community of Rock Run where there are more than a dozen houses that would be destroyed in case of a failure. Further downstream the creek flows into the West Branch of Brandywine Creek. Considering the residential dwellings downstream of the dam, with the potential for loss of life, and the potential for extensive property damage, a "High" hazard classification is justified.

SECTION 6
STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability.

a. Visual Observations. There were no indications observed during the field inspection to indicate that the dam was in an unstable condition. The concrete was assessed to be in relatively good condition with no significant structural cracks or deterioration of the major structural elements. It is noted that facial elements along the downstream side of the structure are in poor condition, in that they are cracked, the reinforcing mesh is exposed, and many of the facial plaster plates are buckling and separated from the buttress walls and top of the dam. Some minor seepage was noted through construction joints and through occasional cracks. However, these seeps are not considered to be signs of imminent instability or distress.

As previously described in Section 3 and shown on Sheet 5a of Appendix B, there was one marshy area located on the right side downstream of the dam. Due to the dense vegetative cover and aerial extent of the seepage and marshy zone, the rate of flow could not be determined. There were no other noted seepage areas located downstream of the dam. Discussions with the operating personnel indicate that this zone has been in this condition as far back as they could remember (approximately 8 years).

b. Design and Construction Data. All available design documentation, calculations and other data pertinent to the structural integrity of the structure were reviewed and assessed for completeness. A detailed listing of this data is included herein as Appendix A and also discussed in Section 2.

Design documentation was, for the most part, relatively complete. There was a complete set of calculations describing the sliding stability, overturning, and hydrostatic forces on the structure. In addition there were several letters of correspondence between the State of Pennsylvania, the designer and the Owner regarding the stability of the structure. Several modifications were performed as a result of suggestions provided by the State. All of these suggestions were on the conservative side and tend to increase the stability of the dam. There was a complete set of specifications and a relatively good set of construction inspection records documenting design changes. As-built plans were limited, but there was sufficient data from the available plans in DER files to delineate the pertinent aspects of this structure. Selected reproductions of these plans are presented in Appendix E as Plates 2 through 4.

Construction photographs were quite comprehensive, covering almost all phases of construction. Photographs depicted the excavation of foundations, the condition of the foundations just prior to concrete placement, and several other aspects of construction. There is also a group of photographs showing the form work, the placement of concrete, and stream diversion. Details pertinent to the

installation of water supply pipes were scarce, but there was sufficient photo documentation to verify how the pipes were installed.

As previously discussed in Section 5, the dam will first overtop at the abutments and probably fail as a result of erosion adjacent to the core walls during the PMF or 0.5 PMF event. Assuming that this deficiency is corrected and that flood waters would pass over the dam, a very approximate stability analysis was performed, as presented in Appendix C-1. The results of this analysis show that the factors of safety for overturning and sliding during the PMF or lesser storms are greater than 1, indicating that the dam will not fail. It is noted that these calculations are approximate and assume that the concrete and steel are in good condition. They also assume that foundation conditions are as described in DER records.

c. Operating Records. There are no operating records maintained for this structure. There are no minimum flow requirements required downstream of the structure. Reservoir level and rainfall records are limited. The only comprehensive records maintained are those of the quantity of water used from the reservoir. There is no maintenance checklist, nor are maintenance records kept.

d. Post-Construction Changes. Since the completion of this dam in 1916, the only major construction activity was the replacement of the concrete cap and other minor concrete repair work performed under the direction of Albright and Friel, Inc. of Philadelphia, Pennsylvania. Since that time, some non-structural facial repair work was performed on the downstream plaster slabs. It is reported by water treatment plant personnel that a series of water supply pipes were installed between the dam and the new water treatment plant. It was also reported that a flow meter was added to the water supply system and a new valve installed to control the water flow.

e. Seismic Stability. The dam is located in Seismic Zone 1. Normally it can be considered that if a dam in this zone is stable under static conditions, it can be assumed safe for any expected earthquake conditions. It is believed that this only applies to earth dams, and not a hollow concrete dam of the "Ambursen" type for Rock Run. Considering the age of this structure and the unknown condition of the main structural elements without a detailed investigation, a seismic stability evaluation could not be performed.

SECTION 7
ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment.

a. Evaluation. The visual inspection, and review of the design and as-built documentation, including photographs, indicates that the dam across Rock Run Creek is in fair condition. The hydrologic and hydraulic computations presented in Appendix C indicate that the dam will only pass 30 percent of the PMF without overtopping. A review of the available structural drawings prepared in 1913 indicates that there is sufficient reinforcing steel in the parapet wall to retain flows up to the top of the wall. The visual inspection noted that the topography on either side of the dam is slightly below the top of the parapet wall. In the event of a flow sufficient to overtop the dam, flow would first occur along both abutments. Since the concrete walls at the ends of the dam are not founded directly on rock, it is expected that erosion would occur and possible catastrophic failure would result during overtopping.

The inspection noted that the pond drain was closed and it is believed that the intake is completely silted in. Attempts were made to exercise the valve, but the leakage was so profuse that the exercise was terminated. Similarly, the lower intake water supply pipe was sealed and the valve could not be operated. Discussions with the water treatment plant representatives indicate that silt is probably at some elevation above the lower intake pipe. Currently, water is only being taken from the top pipe and flows directly into the water treatment facility. This valve is in the open position and remains that way. Based on these observations, it is concluded that it is impossible to drain the reservoir in the event of an emergency using the drainage systems presently in the structure.

b. Adequacy of Information. The available design information was sufficient to evaluate the structure in accordance with Phase I Guidelines provided by the Corps of Engineers. Construction photographs were quite comprehensive, and included practically all aspects of construction. It is noted that the hydraulic calculations were not available, and therefore this aspect of the documentation is considered inadequate. Procedures for evaluating the hydraulic and hydrologic aspects of this facility are discussed in Section 5 and calculations are presented in Appendix C.

c. Urgency. It is concluded that the recommendations presented in Section 7.2 be implemented immediately.

7.2 Remedial Measures.

a. Facilities. It is recommended that the following measures be undertaken by the Owner immediately. The recommendations are presented in order of priority, but does not infer that the latter items are not important.

1. A concrete retaining wall should be installed at each side of the dam to an elevation at least as high as the parapet walls to enable the structure to pass higher flows. This changes the classification of the spillway from "Seriously Inadequate" to "Inadequate", in that the spillway will pass an estimated 68 percent of the PMF.

2. The pond drain system should be rehabilitated so that the reservoir can be drained in the event of an emergency. Alternately, another drainage system should be installed.

3. The spillway and flood storage capacity of the reservoir should be evaluated by a registered professional engineer and the discharge system redesigned to meet current state-of-the-art hydrologic/hydraulic standards.

4. The wet marshy zone located downstream of the dam should be cleared of vegetation and the water collected periodically and checked for changes in flow rates or turbidity.

Items which are not considered critical, but should be attended to in the near future, are the condition of the concrete of the dam's structural elements. A registered professional engineer should evaluate the cross struts between buttresses on the downstream walls. As necessary, these struts should be rehabilitated and recoated with suitable materials to prevent the continual deterioration of the reinforcing steel. Rehabilitation would include the downstream plaster wall and patching of spalled concrete along the spillway and the parapet wall.

b. Operation and Maintenance Procedures. The Owner should develop an inspection checklist together with an inspection and maintenance procedure to insure that all items are properly and periodically inspected, operated and maintained. The outlet supply system should be reassessed to determine if it is necessary that the lower intake pipe be restored to operating conditions.

Because of the downstream population, a formal procedure of observation and warning during periods of high precipitation should be developed and implemented. This procedure should include a method of warning downstream residents that high flows are to be expected along the creek.

APPENDIX

A

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Rock Run Dam (Coatsville Reservoir)

ID # PA 00059

ITEM	REMARKS
AS-BUILT DRAWINGS	15 pages of blueprints ranging from design drawings to a 1949 blueprint of an existing cross-section.
REGIONAL VICINITY MAP	Yes. See Plate 1 of Appendix E.
CONSTRUCTION HISTORY	The DER files contain many progress reports, memos and construction photographs which describe construction thoroughly and in great detail.
TYPICAL SECTIONS OF DAM	The design and construction drawings cntained several cross-sections through the structure.
OUTLETS - PLAN	The design and construction drawings.
DETAILS	
CONSTRAINTS	
DISCHARGE RATINGS	None Available.
RAINFALL/RESERVOIR RECORDS	None.

ITEM	REMARKS
DESIGN REPORTS	1. The application report contains several design items related to hydrology, hydraulics and structures. 2. Structural Computations by Professor McKibben. Complete set of calculations for hollow concrete dam, 7 April 1914.
GEOLOGY REPORTS	Yes. There are several documents in the files which contain geologic data. A detailed list is contained on sheet 4a.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	DER files contain several documents covering these features of design in varying degress of completeness. The text of the report describes each feature in more detail.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	The "Report Upon the Application" covers the investigation aspects and the progrss reports cover the field aspects of the construction.
POST-CONSTRUCTION SURVEYS OF DAM	None known.
BORROW SOURCES	Concrete dam. There is very little earth work associtated with this structure. (less than 31,000 cubic yards placed at each end of the dam)

ITEM	REMARKS
MONITORING SYSTEMS	*None*
MODIFICATIONS	*No major modifications were made after construction.*
HIGH POOL RECORDS	*None available.*
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	*No significant studies are known.*
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	*None*
MAINTENANCE OPERATION RECORDS	*None*

ITEM	REMARKS
SPILLWAY PLAN	
SECTIONS DETAILS	*See Appendix E.*
OPERATING EQUIPMENT PLANS & DETAILS	*See Appendix E for section of water supply intake system.*
MISCELLANEOUS	1. *"Specifications for Construction of Repairs to Overflow Weir and Miscellaneous Repairs to Rock Run Dam" for City of Coatsville, Pa., by Albright and Friel, Inc., Philadelphia, Pa., 1949.* 2. *"Permit" dated 9 October 1916, to alter ends of concrete walls of dam.* 3. *"Application" dated 30 September 1949 to replace concrete weir cap.* 4. *"Supplementary Report Upon the Application" 25 March 1915 by the State of Pennsylvania.* 5. *"Supplementary Report Upon the Application" 25 October 1915, by George S. Beal, State of Pennsylvania.* 6. *Foundation Approval forms signed by State Representatives.* 7. *Miscellaneous sketches of "As-Built" foundation sections.* 8. *Progress reports throughout construction phase.* 9. *12 February 1919 Inspection Report by Frederick P. Stearns, Consulting Engineer.* 10. *State Inspection Reports, 1920 through 1970.*

APPENDIX

B

CHECK LIST
VISUAL INSPECTION
PHASE I

Name Dam *Rock Run Dam* County *Chester* State *Pennsylvania* National ID # *PA 00059*

Type of Dam *Concrete-Hollow Core (Ambursen)* Hazard Category *I (High)*

Date(s) Inspection *31 July 1978* Weather *Rain & Cloudy* Temperature *60-70's°F*

Pool Elevation at Time of Inspection *485* M.S.L. Tailwater at Time of Inspection *454±* M.S.L.

Inspection Personnel:

Mary Beck (Hydrologist)	*John Boschuk, Jr. (Geotech/Civil)*
Vince McKeever (Hydrologist)	*John H. Frederick, Jr., (Geotechnical)*
Brady Bisson (Geotechnical)	

John Boschuk, Jr. Recorder

Remarks:

Messrs. Steve Powers, Superintendent; Fred Reed, Operator; and Miller, County of Coatsville were on site and provided information and assistance during the inspection.

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
ANY NOTICEABLE SEEPAGE	*Yes. Small seeps and several wet areas were noted along cracks and construction joints along the upstream section of the dam. At several locations calcium carbonate stalactites were noted and many of these were stained with iron oxide.*	
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	*Both abutments appeared to be in good condition with no signs of leakage or excessive seepage.*	
DRAINS	*Drains between buttress walls were clear and draining well. Drains along the donwstream chute were clear and functioning well.*	
WATER PASSAGES	*N/A*	
FOUNDATION	*There were no external signs of distortion, rotation, settlement or other phenomena to indicate any potential instability of the foundation.*	

CONCRETE/MASONRY DAMS

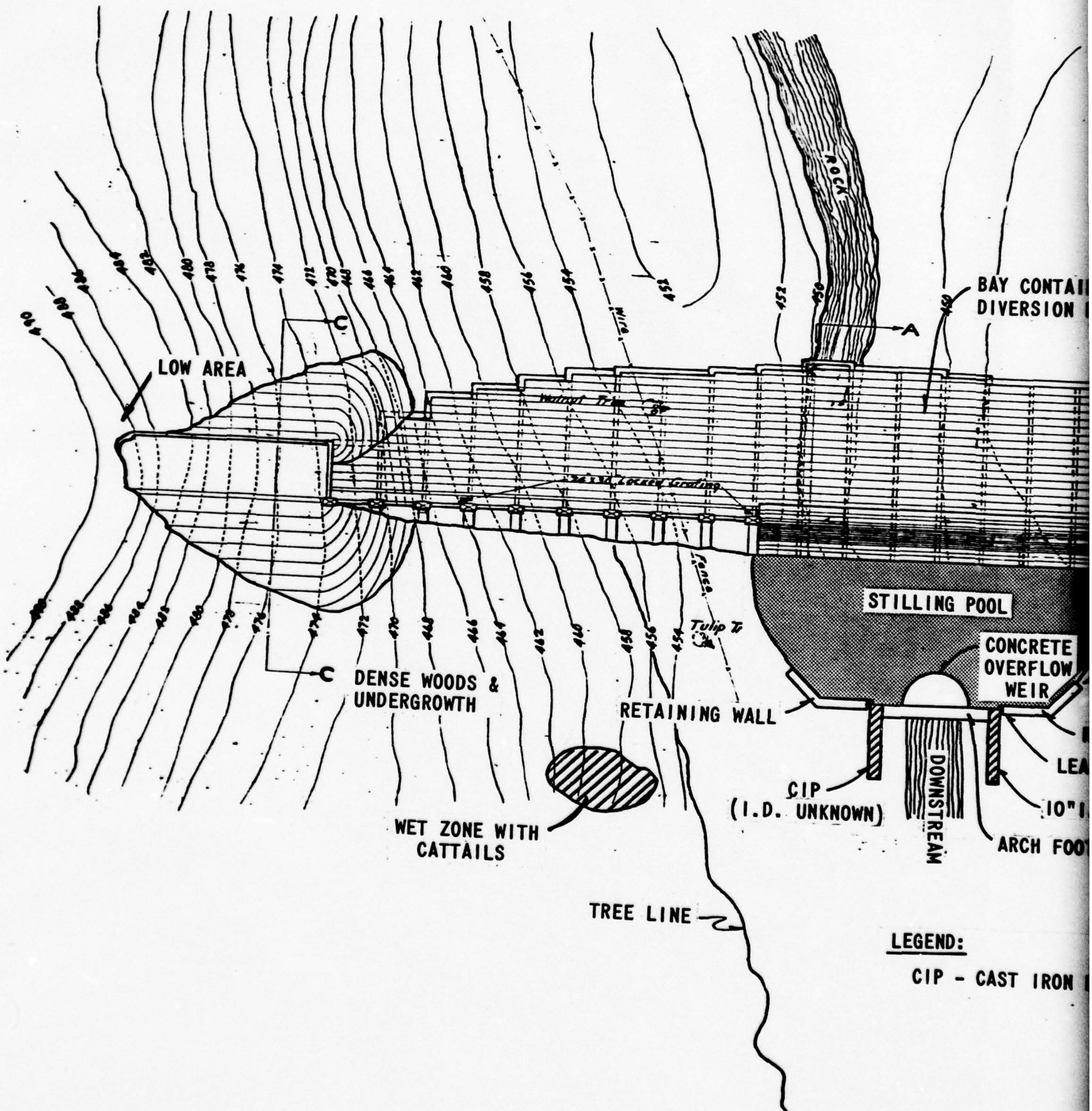
VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	*Both the upstream and downstream sections of the spillway contain spalled areas with exposed reinforcing bars. Some areas were patched and, in general, these areas were in good condition. Cracks were also noted on both the upstream and downstream sections. There was a hole in the downstream facing plate which measured 1 1/2 feet x 6 feet. This hole does not effect the stability of the dam.*	
STRUCTURAL CRACKING	*Some minor, hairline, cracking was noted on the load-bearing slabs particularly near the top of the dam on the upstream side. These cracks were noted under the slab when viewed from the inspection gallery. Cross-beams between buttresses on the downstream side were often spalled and reinforcing steel was exposed. There were no significant cracks found on these beams.*	
VERTICAL AND HORIZONTAL ALIGNMENT	*No signs of movement of the load-bearing portions of the structure. However, the downstream facial plates are cracked and movements in the form of separation between structural elements and the facing were noted (the facing moved).*	
MONOLITH JOINTS	*None. Ambursen dam; hollow concrete spillway with an interior inspection gallery.*	
CONSTRUCTION JOINTS	*Several construction joints were leaking on the upstream side with occasional calcium carbonate deposits.*	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	N/A	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	N/A	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	N/A	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	N/A	
RIPRAP FAILURES	N/A	

EMBANKMENT

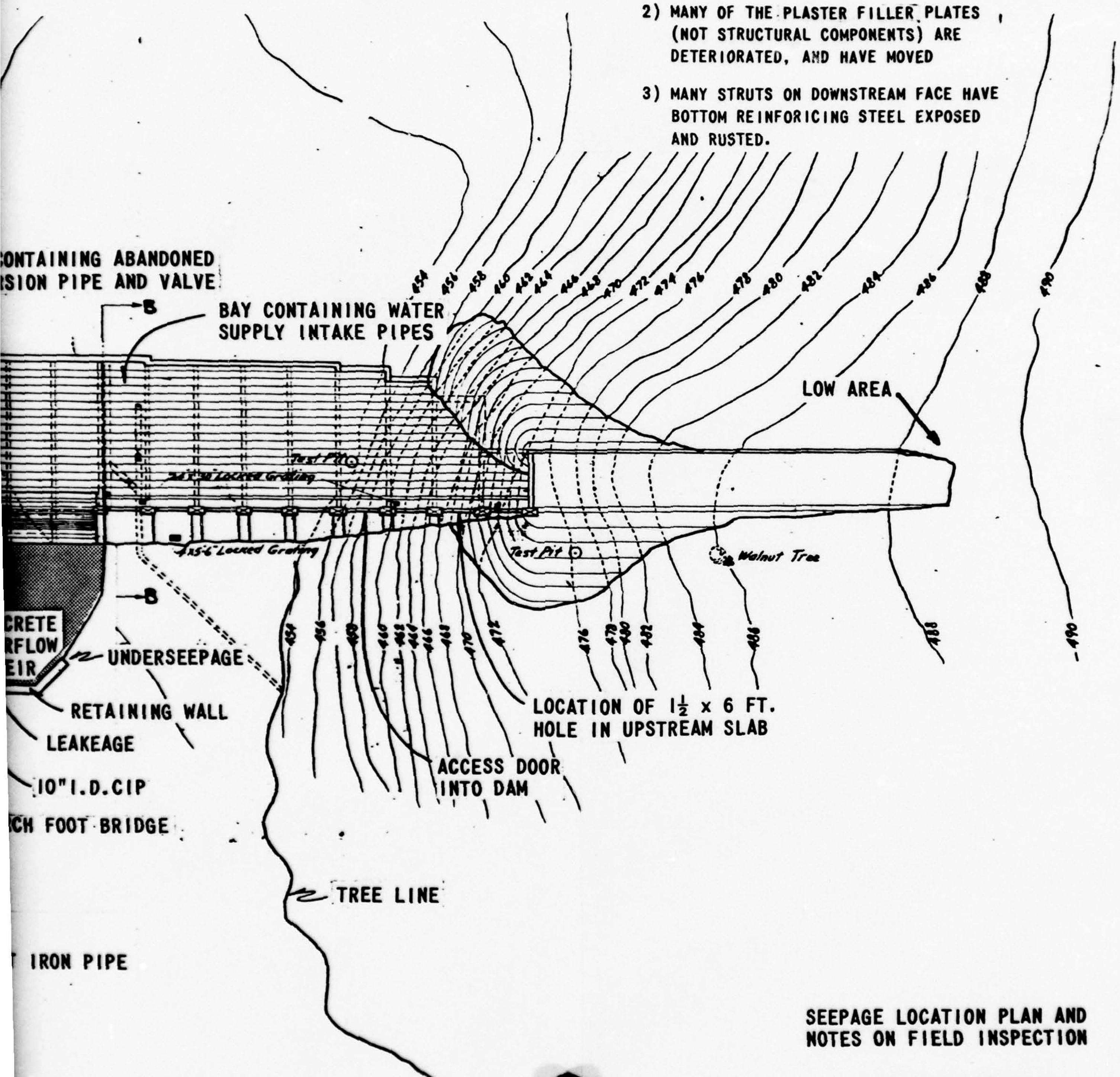
VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	N/A	
ANY NOTICEABLE SEEPAGE	N/A	
STAFF GAGE AND RECORDER	N/A	
DRAINS N/A		


ROCK
A
C
LOW AREA
Walnut Tree
36x30 Locked Grating
Fence
Tulip Tr
STILLING POOL
CONCRETE
OVERFLOW
WEIR
DENSE WOODS &
UNDERGROWTH
RETAINING WALL
CIP
(I.D. UNKNOWN)
DOWNSTREAM
WET ZONE WITH
CATTAILS
TREE LINE
LEGEND:

NOTES: 1) MANY UPSTREAM CONSTRUCTION JOINTS LEAK. SOME HAVE HAIRLINE CRACKS.

2) MANY OF THE PLASTER FILLER PLATES (NOT STRUCTURAL COMPONENTS) ARE DETERIORATED, AND HAVE MOVED

3) MANY STRUTS ON DOWNSTREAM FACE HAVE BOTTOM REINFORICING STEEL EXPOSED AND RUSTED.



SEEPAGE LOCATION PLAN AND NOTES ON FIELD INSPECTION

SHEET 5a OF 11

OUTLET WORKS

(WATER SUPPLY PIPES)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	*N/A Cast iron pipes embedded in the upstream portion of the dam constitute the components of the intake/outlet works.*	
INTAKE STRUCTURE	*This consists of 4-inch cast iron pipes embedded in the concrete wall of the dam. The pipes were in reasonably good condition, but the valves were inoperable. A valve was installed several years ago to control water from the upper intake. Attempts were made to exercise the valve, but it began to leak and no further attempts were made. All of the valves were rusty and not well maintained. Water supply valves are all in the open position. All other valves are presumed closed (based on the Owners statements).*	
OUTLET STRUCTURE	*All pipes feeding water to the treatment plant are buried and could not be inspected.*	
OUTLET CHANNEL	*N/A*	
EMERGENCY GATE	*Valves were rusty, leaking and inoperable.*	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	*The 117-foot long weir has a 0.2±foot notch in the center, water was flowing smoothly through notch. The concrete spillway cap was dislocated forward, or, the downstream spillway slab moved upstream. This weir should be repaired.*	
APPROACH CHANNEL	*N/A*	
DISCHARGE CHANNEL	*Discharge from dam enters stilling pool formed by a semi-circular weir, 25.5 feet in diameter, and flows under a concrete slab foot-bridge. Weir and foot-bridge appeared in good condition. There are several spalled areas of concrete which do not significantly affect the stability of the bridge.*	
BRIDGE AND PIERS	*N/A*	
PARAPET WALLS & RETAINING BUTTRESS SECTIONS	*The parapet walls, although spalled, are in reasonably good condition and structurally sound. The butress sections are in good condition with only hairline cracking of the load bearing diaphram walls. Displacement of the non-load bearing downstream facing walls was observed. Reinforcing steel was exposed at several locations and several sections were assessed to be unstable. These downstream facing walls should be repaired.*	

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL *N/A*		
APPROACH CHANNEL *N/A*		
DISCHARGE CHANNEL *N/A*		
BRIDGE AND PIERS *N/A*		
GATES AND OPERATION EQUIPMENT *N/A*		

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS *None*		
OBSERVATION WELLS *None*		
WEIRS *None*		
PIEZOMETERS *None*		
OTHER *None*		

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	*Reservoir side slopes are flat to moderate, well vegetated with grass and trees. Very little debris observed.*	
SEDIMENTATION	*Sedimentation has occurred, especially at the upper end of the reservoir, such that the normal volume of the reservoir has been reduced. Sedimentation has little or no effect on available flood storage. The upper end of the reservoir was dredged in the early 1970's. The reservoir immediately upstream of the dam was inspected in 1972± and observed that the middle intake was covered with silt and not capable of functioning.*	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	*Rock Run flows through a 30-foot by 13-foot bridge under Water Works Road about 220 feet downstream of the dam and then through open/farm and some wooded areas for the next 1.5 miles. About 1.4 miles downstream a small unnamed stream enters Rock Run and the stream passes under U.S. 30 By-Pass. If the dam failed, large trees and debris could take this bridge out.*	
SLOPES	*The stream banks are generally low, well vegetated and fairly stable. The valley gradient is approximately 0.9 percent.*	
APPROXIMATE NO. OF HOMES AND POPULATION	*One or two houses, in the first 1.5 miles, are subject to damage if the dam fails. Approximately two miles downstream of the dam Rock Run passes through a community of the same name where more than one dozen houses would be destroyed in case of dam failure.*	

APPENDIX

C

Rock Run Dam
CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: *Predominantly open farmland, 10-15 percent residential.*

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): *485.0 (1019 Acre-Feet).*

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): *489.0 (1252 Acre-Feet).*

ELEVATION MAXIMUM DESIGN POOL: ------

ELEVATION TOP DAM: *489*

SPILLWAY

a. Elevation *485.0*

b. Type *Concrete ogee shaped.*

c. Width *117 feet.*

d. Length ---------

e. Location Spillover *At the center of the dam.*

f. Number and Type of Gates *None.*

OUTLET WORKS:

a. Type *Multilevel pipe intake.*

b. Location *Upstream face of dam.*

c. Entrance inverts *2-18 inch CIP @ 463.0 and 473.0.*

d. Exit inverts *Common 18 inch pipe into Water Treatment Plant.*

e. Emergency draindown facilities *24 inch CIP, inlet at 452.75.*

HYDROMETEOROLOGICAL GAGES:

a. Type *None.*

b. Location -----

c. Records ----

MAXIMUM NON-DAMAGING DISCHARGE: *Undetermined.*

DAM SAFETY ANALYSIS
HYDROLOGIC/HYDRAULIC DATA

Date: 8/16/78
By: MFB
Sheet: 2 of 10

DAM Rock Run Dam Nat. ID No. PA 059 DER No. 15-4

	ITEM/UNITS	Permit/Design Files (A)	Calc. from Files/Other (B)	Calc. from Observations (C)
1.	Min. Crest Elev., ft.	489.0		
2.	Freeboard, ft.			
3.	Spillway(1) Crest Elev, ft.	485.0		
3a.	Secondary(2) Crest Elev, ft.			
4.	Max. Pool Elev., ft.	488.0		
5.	Max. Outflow(3), cfs	2310.0		
6.	Drainage Area, mi^2	5.32/6.47		5.69
7.	Max. Inflow(4), cfs	4772		
8.	Reservoir Surf. Area, Acre	61.0		58.2
9.	Flood Storage(5), Acre-Feet			
10.	Inflow Volume, Acre-Feet	779.3		

Reference all figures by number or calculation on attached sheets:

Example: 3A - Drawing No. xxx by J. Doe, Engr., in State File No. yyyy.

NOTES:

(1) Main emergency spillway.

(2) Secondary ungated spillway.

(3) At maximum pool, with freeboard, ungated spillways only.

(4) For columns B, C, use PMF.

(5) Between lowest ungated spillway and maximum pool.

Date: 8/16/78
By: MFB
Sheet: 9 of 10

HYDROLOGIC/HYDRAULIC CALCULATIONS (cont.)

Item (from Sheet 2)	Source
1A, 3A	Design Drawing dated 1913
4A, 5A, 6A, 7A, 8A, 10A	Application Report dated Jan. 30, 1914
6A, 8A	USGS Maps Wagontown (1969) Coatsville (1973) Honey Brook (1974) Parkesburg (1973)

Classification (Ref. Recommended Guidelines for Safety Inspection of Dams)

1. The hazard potential is rated as HIGH as there would be loss of life if the dam failed.

2. The size classification is INTERMEDIATE based on its height of 42 ft. and 1019 Ac-Ft. normal pool volume

3. The spillway design flood, based on size and hazard classification, is the probable maximum flood (PMF).

Hydrologic and Hydraulic Analysis

1. Spillway Capacity

   Original Data

   Q = 2310 cfs H = 3 ft. L = 117 ft

   Q = 1253 cfs H = 2 ft.

   the above indicates the use of C = 3.80 in the equation $Q = CLH^{3/2}$ and is acceptable by current practice

   Therefore: Q = 3560 cfs H = 4 ft. top of dam

   Q = 8230 cfs H = 7 ft. top of parapet wall

2. Spillway Design Flood

   Original Data

   Rainfall = 5.48 inches in two hours

   Runoff = 50% (average)

   Peak Inflow = 4772 cfs

   Evaluation Data

   Rainfall (PMP) = 26 inches in 6 hours over 10 sq. miles (Ref. TP-40)

   Runoff = 90% assumed

   Peak Inflow = Information from Corps of Engineers, Baltimore District indicates the peak inflow = 1475 cfs/mile^2

   Area = 5.69 sq. mile from USGS maps

   therefore peak inflow = 8393 cfs

BY MEB DATE 8/21/78 SUBJECT Rock Run Dam SHEET 5 OF 10

CHKD. BY ____ DATE ____ Hydrology / Hydraulics JOB No. ____

Approximatel 8 miles from Rock Run Dam is Marsh Creek Dam which was inspected under the National Dam Safety Program. Marsh Creek Dam hydrologic design was according to Soil Conservation Service criteria.

Marsh Creek Data

Drainage Area	20 sq. miles
Peak PMF Inflow	36,000 cfs
Inches of Runoff	24.8

Peak PMF Inflow transfered to Rock Run Watershed

$$Q_I = \left(\frac{5.69}{20}\right)^{0.8} 3600 = 13169 \text{ cfs}$$

USE 13200 cfs

Therefore, use of Marsh Creek watershed is not unconservative and peak PMF = 13,200 cfs will be used.

Volume of Runoff, V_I =

$$\frac{24.8}{12} \cdot 5.69 \cdot 640 = 7525 \text{ Ac-Ft.}$$

3. Overtopping Potential

Using short-cut method, see sheets 9 & 10

With reservoir level at top of dam

Q_o = 3560 cfs

V_S = available flood storage = 233 Ac-Ft (sheet 7)

V_R = required storage

$$= \left(1 - \frac{Q_o}{Q_I}\right) V_I$$

$$= \left(1 - \frac{3560}{13200}\right) 7525 = 5495 \text{ Ac-Ft} > V_S = 233 \text{ Ac-Ft.}$$

for 0.5 PMF

Q_I = 13200/2 = 6600 cfs

V_I = 7525/2 = 3762 Ac-Ft

$$V_R = \left(1 - \frac{3560}{6600}\right) 3762 = 1733 \text{ Ac-Ft} > V_S$$

Therefore, the dam will be overtopped by 0.5 PMF

With reservoir level at top of parapet wall

$Q_o = 8230$ cfs

$V_S = 403$ Ac-Ft.

$V_R = \left(1 - \frac{8230}{13200}\right) 7525 = 2833$ Ac-Ft $< V_S$

for 0.5 PMF

$Q_I = 6600$ cfs

as $Q_o > Q_I$, dam would pass 0.5 PMF

Percentage of PMF passed with reservoir level at top of parapet wall

$Q_I = X\ 13200$

$V_I = X\ 7525$

$403 = \left(1 - \frac{8230}{X 13200}\right) X 7525$

$X = 68\%$

Percentage of PMF passed with reservoir level at top of dam

$Q_o = 3560$

$233 = \left(1 - \frac{3560}{13200X}\right) 7525X$

$X = 30\%$

Depth of flow over parapet wall during the PMF storm. (w/ dikes to prevent flow around dam)

If depth of flow is 1.5 ft. then:

$V_S \sim 460$ Ac-Ft

$Q_o = 117 \cdot 3.8 \cdot 8.5^{3/2} = 11{,}018$ cfs

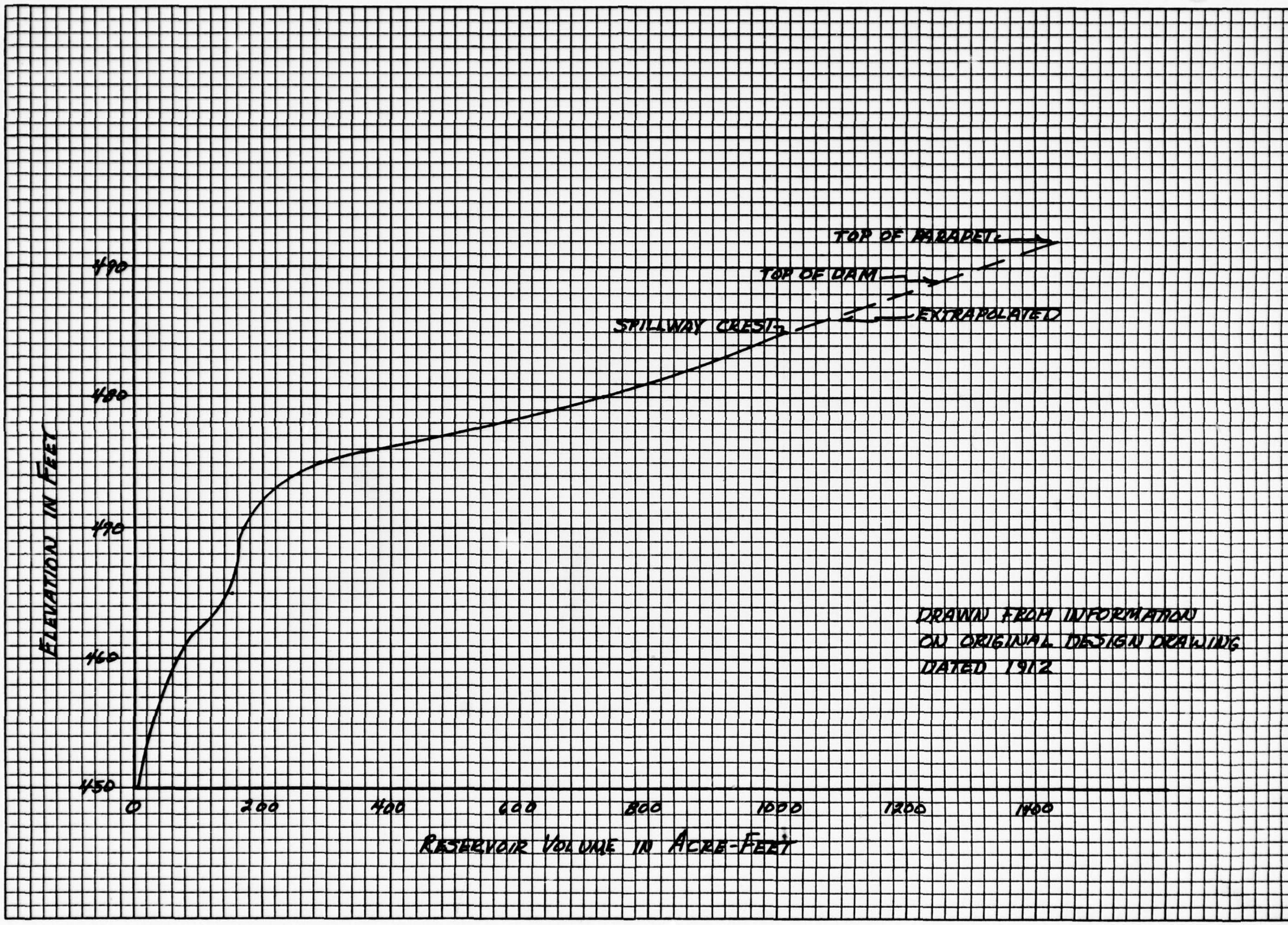
$= 276 \cdot 2.8 \cdot 1.5^{3/2} = 1420$

12,438 cfs total

(assume flow over parapet wall only, length = 276 ft.
C = 2.8 as estimated using Brater & King, Handbook of Hydraulics, Table 5-3)

$V_R = \left(1 - \frac{12438}{13200}\right) 7525 = 434$ Ac-Ft. $\sim V_S$

Therefore, depth of flow over parapet wall ~1.5 ft.


TOP OF PARAPET
TOP OF DAM
SPILLWAY CREST
EXTRAPOLATED
490
480
470
460
450
ELEVATION IN FEET
0
200
400
600
800
1000
1200
1400
RESERVOIR VOLUME IN ACRE-FEET
DRAWN FROM INFORMATION
ON ORIGINAL DESIGN DRAWING
DATED 1912

BY MFB DATE 8/20/78 SUBJECT Rock Run Dam SHEET 8 OF 10

CHKD. BY ____ DATE Rev 9/18/78 Hydrology / Hydraulics JOB No. ____

Estimated reservoir surface level during a 0.5PMF event (w/ dikes to prevent flow around dam)

For 0.5PMF

$Q_I = 6600 \text{cfs}$

$V_1 = 3762.5 \text{ cfs}$

IF reservoir surface ~ 490.6

$Q_0 = 3.8 \cdot 117 \cdot 5.6^{3/2} = 5890 \text{ cfs}$

$V_2 = \left(1 - \frac{5890}{6600}\right) 3762.5 = 403 \sim V_3 = 350 \text{ Ac-Ft.}$

Therefore, reservoir level will be about 490.6 ft during 0.5PMF (about 1.6 ft over top of dam)

4 Spillway Adequacy - See text, Section 5, for discussion.

5. Downstream Conditions

About 220 ft. downstream of Dam Rock Run passes under 13 x 30 ft. bridge opening.

Estimate capacity of bridge by Manings Equation

$v = \frac{1.49}{n} r^{2/3} s^{1/2}$

$n = 0.035$ (field estimate)

r = area ÷ wetted perimeter, assume flowing full

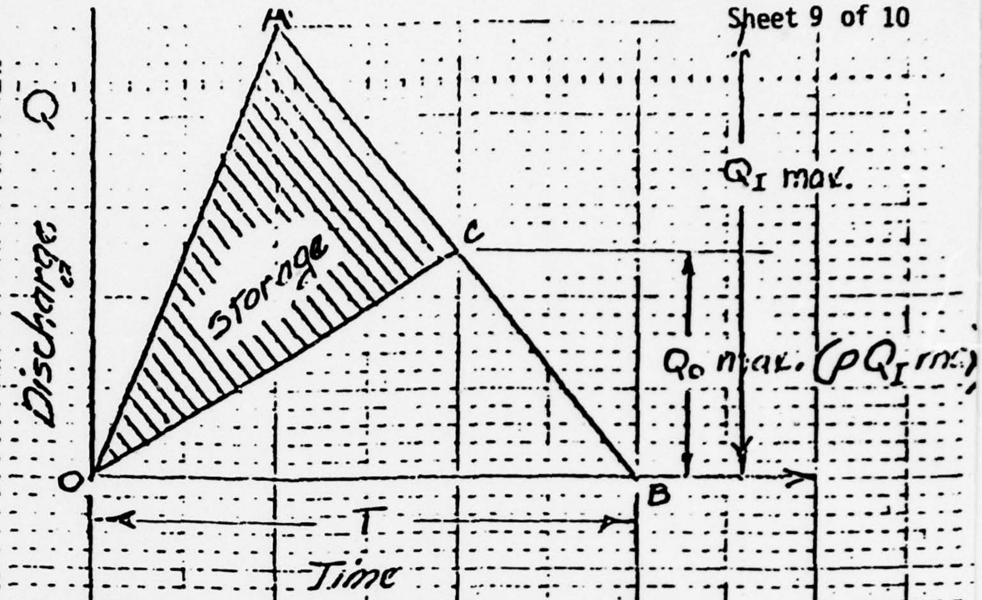
s = channel slope

drop along channel bottom between bottom of stilling basin and bridge is about 1.5 ft, therefore, $s_o = 1.5/200 = 0.0075$

$v = \frac{1.49}{0.035} \left(\frac{390}{56}\right)^{2/3} 0.0075^{1/2}$

$= 13.4$ ft./sec

$Q = aV = 5243$ cfs, at which time the bridge approach to the east will be flooded out.



PURPOSE: Establish relationship between maximum spillway discharge and storage required to pass flood hydrograph without exceeding maximum pool level.

$$\frac{\Delta AOC}{\Delta AOB} = \frac{\Delta AOB - \Delta COB}{\Delta AOB} = 1 - \frac{\Delta COB}{\Delta AOB}$$

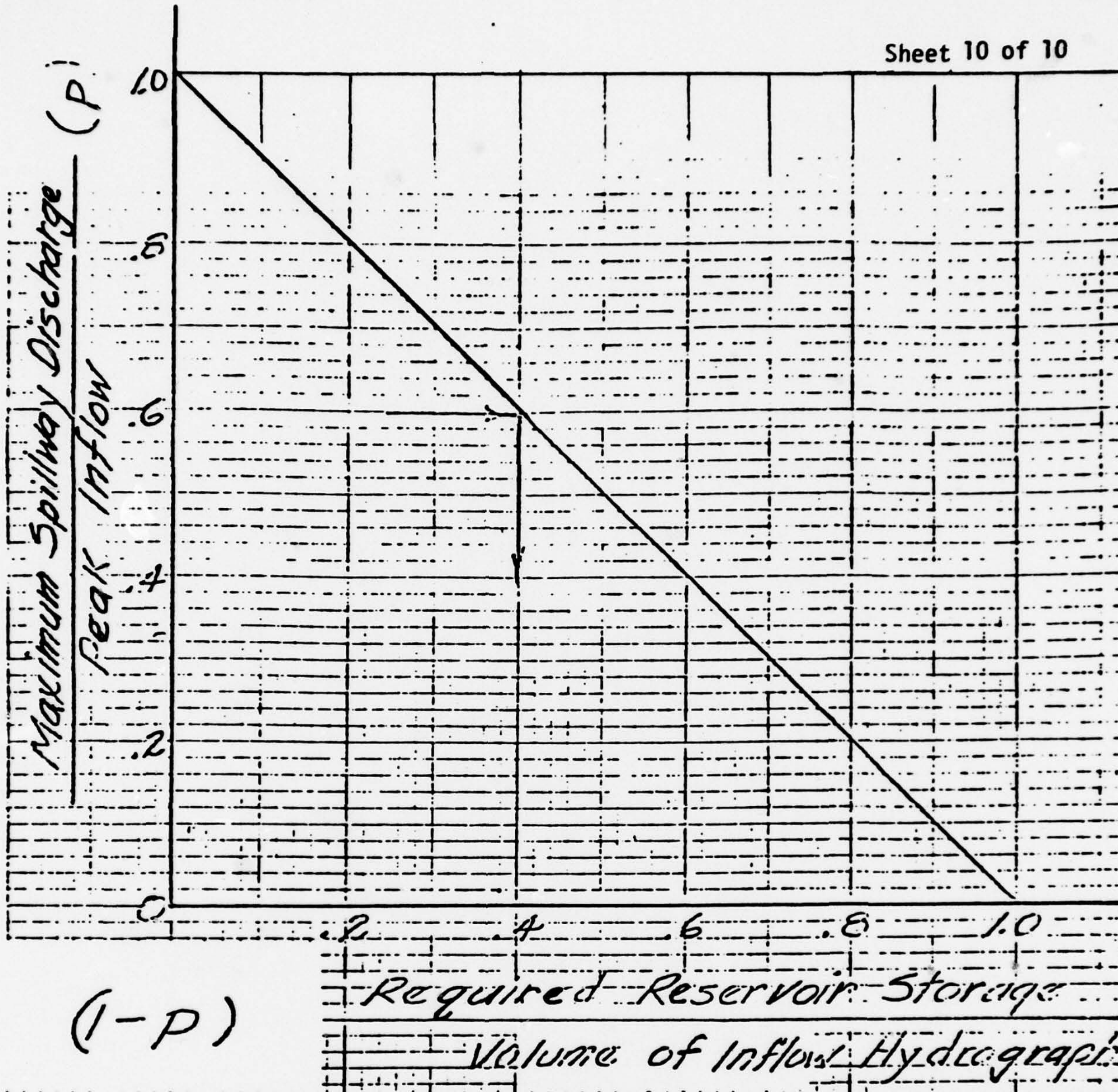
$$\frac{\Delta AOC}{\Delta AOB} = 1 - \frac{T \rho Q_I \max/2}{T \cdot Q_I \max/2} = 1 - \rho$$

$$\Delta AOC = (1-\rho)\,\Delta AOB \quad \text{where } 0 \le \rho \le 1.0$$

ρ	Δ AOC
1.00	0
0.75	0.25 Δ AOB
0.50	0.50 Δ AOB
0.25	0.75 Δ AOB
0	1.00 Δ AOB

REFERENCE

PRELIMINARY ENGINEER TECHNICAL LETTER NO. 1110-2-
25 January 1978



Steps to obtain required reservoir to pass inflow hydrograph without overtopping dam.

1. Obtain maximum spillway discharge
2. Develop inflow hydrograph
3. Compute relationship of maximum spillway capacity to peak inflow
4. Read relationship of required reservoir storage to volume of inflow hydrograph from curve

APPENDIX

C - 1

BY DC DATE 9/21/78 SUBJECT STABILITY ANALYSIS SHEET 1 OF 10

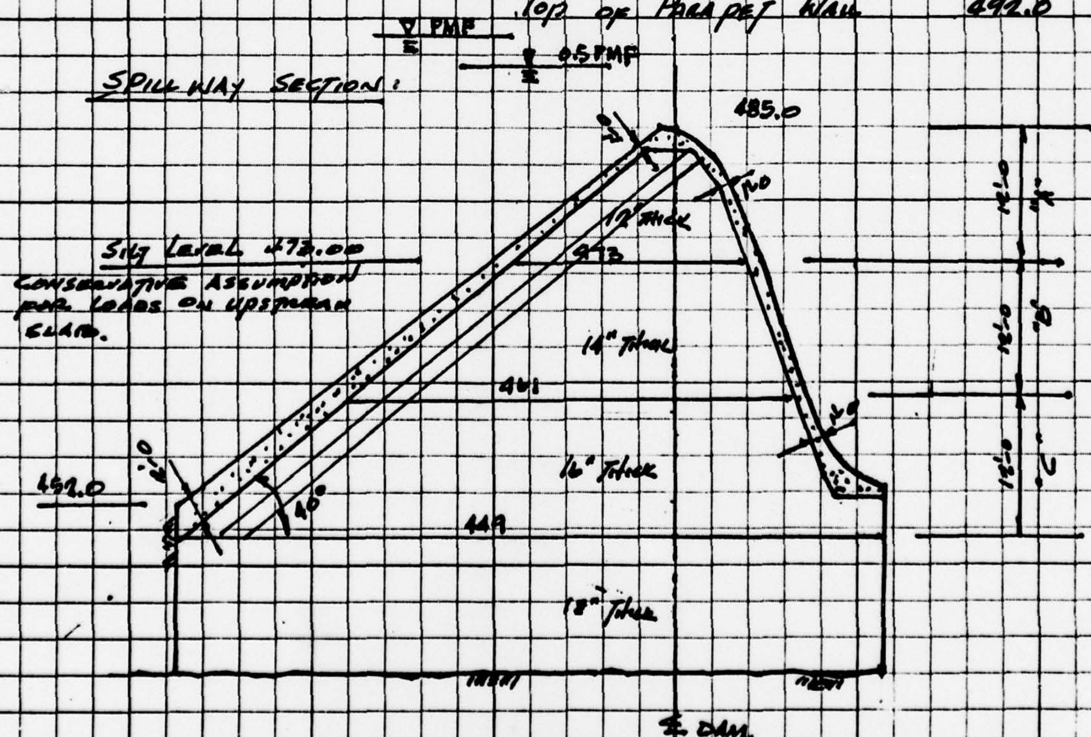
CHKD. BY [illegible] DATE 9/25/78 JOB No. ROCK-RUN DAM

OBJECTIVE :- DETERMINE STABILITY OF ROCK-RUN DAM WHEN OVERTOPPED BY 0.5 PMF & PMF

ELEV. OF WATER AT 0.5 PMF	=	490.6'
ELEV. OF WATER AT PMF	=	493.5'
ELEV. OF SILT (ASSUMED)	=	473.0
ELEV. OF BOTTOM OF RESERVOIR	=	462.0

GEOMETRY OF DAM :	Top of SPILLWAY	485.0
	Top of DAM	489.0
	Top of PARAPET WALL	492.0

SPILLWAY SECTION:



TYPICAL BUTTRESS AT SPILLWAY — BAY SIZE 15' c/c.

NOTE: THIS IS THE SMALLEST SECTION OF DAM. THUS THIS SECTION GOVERNS STABILITY OF THE DAM.

BY DC DATE 9/21/78 SUBJECT

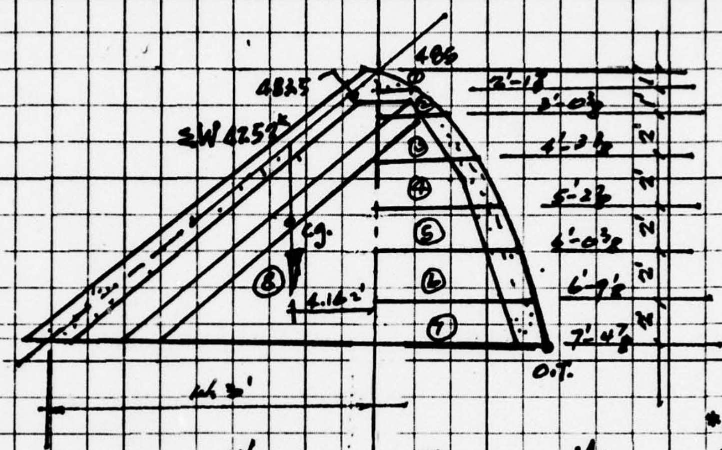
SHEET 2 OF 10

CHKD. BY DATE

JOB No. ROCK RUN DAM

STABILITY OF BUTTRESS

WALL "A" : 12" THICK



*CENTROID OF TRAPEZOID

ELEMENTS	AREA	WT.	$\bar{x}$	$M_{℄}$
①	$(2'\text{-}1\tfrac{3}{8} \times 1')\tfrac{1}{2} = 1.06^{\square'}$	0.159^k	1.410	0.224
②	$\left(\frac{2'\text{-}1\frac{3}{8} + 3'\text{-}0\frac{3}{8}}{2}\right)\times 1 = 2.576$	0.386^k	1.463*	0.565
③	$\left(\frac{3'\text{-}0\frac{3}{8} + 4'\text{-}3\frac{3}{8}}{2}\right)\times 2 = 7.31^{\square'}$	1.10^k	2.06*	2.266
④	$\left(\frac{4'\text{-}3\frac{3}{8} + 5'\text{-}2\frac{3}{8}}{2}\right)\times 2 = 9.52^{\square'}$	1.428^k	2.485*	3.549
⑤	$\left(\frac{5'\text{-}2\frac{3}{8} + 6'\text{-}0\frac{3}{8}}{2}\right)\times 2 = 11.27^{\square'}$	1.691^k	2.878*	4.867
⑥	$\left(\frac{6'\text{-}0\frac{3}{8} + 6'\text{-}9\frac{3}{8}}{2}\right)\times 2 = 12.79^{\square'}$	1.919^k	3.243*	6.223
⑦	$\left(\frac{6'\text{-}9\frac{3}{8} + 7'\text{-}4\frac{3}{8}}{2}\right)\times 2 = 14.17^{\square'}$	2.123^k	3.574*	7.588
⑧	$14.3 \times \frac{12}{2} = 85.8^{\square'}$	12.871^k	-4.767'	-61.356

HAUNCHES VOL. 10.5449 □' × 16.335 = 172.25 ft³

WT. 25.843^k

$\bar{x}$ -6.257'

M = -161.690

ΣW = 47.52^k

ΣM -197.764

$\bar{x}$ = -4.162'

OW OF BUTTRESS. $M_R = 47.52 \times (4.162 + 7.406) = 549.71'^k$ (11.568)

BY DC DATE 1/21/78 SUBJECT

SHEET 3 OF 10

CHKD. BY DATE

JOB No. ROCK RUN DAM

WT. OF WATER ABOVE THE DAM:- @ 0.5 PMF +490.6, $\gamma_{WATER} = 62^{\#}/ft^3$

$5.6' \times 14.3' \times 62.4 = 5^k \times \left(\frac{14.3}{2} + 7.403\right)' = 72.78'^k \times 15' = 1091.7'^k$

$14.3 \times \frac{12}{2} \times 62.4 = 5.35 \times \left(\frac{2 \times 14.3}{3} + 7.403\right) = 90.6'^k \times 15' = 1359.14'^k$

$2450.84'^k$

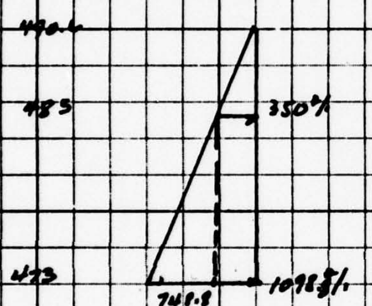
TOTAL M_R = BUTTRESS = $549.71'^k$

WATER = $2450.84'^k$

$3000.55'^k$

NOTE:- WT. OF SLABS NEGLECTED.

OVERTURNING MOMENT:-

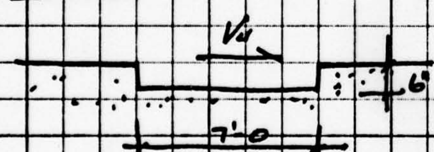


$H = \frac{1098 \times 17.6}{2} \times 15 = 145.04$

$M_{OT} = 145.04 \times \frac{17.6}{3} = 850.9'^k$

$F.S. = \frac{M_R}{M_{OT}} \quad \frac{3000.55}{850.9} = \underline{\underline{3.53}}$ FACTOR OF SAFETY OF OVERTURNING @ 0.5 PMF. AT ELEV. 473

SLIDING :-



USING ACI - 318-71 LOAD FACTOR FOR HYDROSTATIC LOADS 1.4

V_u @ SHEAR KEY:- $V_u = 1.4 \times 145 = 203^k$

STRESSES AS BM SHEAR $v = \frac{VQ}{Ib} = \frac{3}{2}\frac{V}{A}$

$v_u = \frac{1.5 \times 203}{(.85)\ 84 \times 12} = \underline{\underline{355\ psi}}$

ACCORDING TO FERGUSON, "REINFORCED CONC. FUNDAMENTALS"; ALLOWABLE SHEAR IN KEYS IS SIMILAR TO STIRRUP WHICH CODE LIMITED TO $8\sqrt{f'_c}$

ASSUME $f'_c = 3000$ psi (VERY CONSERVATIVE)

$V_{U_{MAX}} = 8\sqrt{3000} = 438$ psi > 355 . $- v_c = 245$ psi ($5 \to 110$ psi) $\therefore$ OK!

BY DC DATE 9/21/78 SUBJECT

CHKD. BY DATE

JOB No. ROCK RUN DAM

BEARING:- $f_{B'R'G} = \frac{203}{12 \times 6} = 2.82\ ksi > 0.85 \times .7 \times 3 = 1.785\ ksi$

∴ TRANSFER OF FORCE ALSO REQUIRES FRICTION BTWN WALL "A" & WALL "B" BELOW.

ASSUME THE KEY WOULD DEVELOP MAX B'R'G.

$H_{B'R'G.} = 1.785 \times 72 = 128.5^k$

∴ $(203 - 128.5) = 74.5^k$ MUST BE DEVELOPED THRU FRICTION.

$\mu = 1.0$ FOR HARDEN. CONCRETE AGAINST CONCRETE
(ACI - 318 - 71)
(CHAPTER 11)

TOTAL WT. OF WALL & WATER

$(5^k + 5.35^k)15 + 47.52 = 202.8^k \times 1.4 = 283.9^k$

$F_{MAX} = 283.9 > 74.5^k$ ∴ OK!

$F.S.^* = \frac{283.9}{74.5} = \underline{\underline{3.81}}$ FACTOR OF SAFETY AGAINST SLIDING AT 0.5 P.M.F. @ JT. ELEV. 473

WT. OF WATER ABOVE THE DAM @ PMF. 493.5

$8.5' \times 14.3' \times 15 \times 62.4 = 113.77^k \times (\frac{14.3}{2} + 7.403) = 1946.75^{'k}$

TOTAL MOMENT (RESISTING) = 1946.76 + 1359.14 + 549.71

$= \underline{\underline{3855.6}}^{'k}$

HYDROSTATIC PRESSURE

$H = \frac{62.4 \times 20.5^2}{2} \times 15' = 196.68^k$

$M_{OT} = 196.68 \times \frac{20.5}{3} = 1343.96^{'k}$

*SEE NOTE ON SHT B.

BY DC DATE 9/21/78 SUBJECT SHEET 5 OF 10

CHKD. BY DATE JOB No. ROCK RUN DAM .

OVERTURNING :- $F.S. = \frac{3855.6}{1343.96} = 2.87$ ✓

FACTOR OF SAFETY OF OVERTURNING AT PMF @ JT. EL. 473.

SLIDING :- $H_u = 1.4 \times 196.68^k = 275.4^k$

$V_u = \frac{3}{2} \frac{275.4}{84 \times 12 \cdot .85} = 482$

$V_u - V_c = 372$ psi < 438 psi ∴ ok!

TOTAL D.W. OF STRUCTURE + LOADS = 80.25 + 113.77 + 42.52
= 241.54^k

$F_{MAX} = 241.54^k \times 1.4 = 338.16^k$

NET F = 275.4 − 128.5 = 146.9^k < 338.16

$F.S.^* = \frac{338.16^k}{146.9^k} = \underline{\underline{2.3}}$ ✓

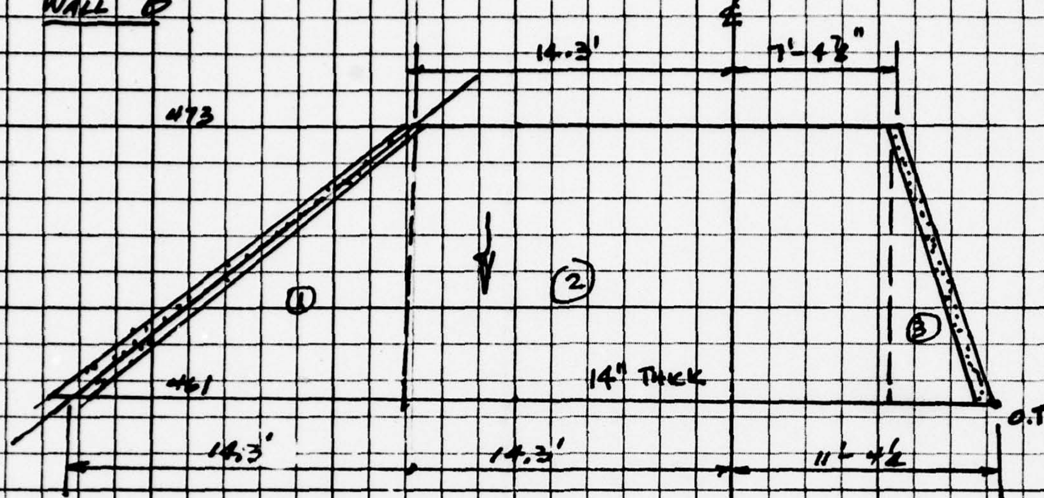
FACTOR OF SAFETY AGAINST SLIDING @ PMF. @ JT. ELEV. 473

* SEE NOTE ON SHT 8

BY DC DATE 9/22/78 SUBJECT SHEET 6 OF 10

CHKD. BY DATE JOB No. ROCK RUN DAM

WALL "B"



ELEMENT	AREA	WT.	$\bar{x}$	$M_{\text{¢}}$
①	$\frac{14.3 \times 12}{2} = 85.8\square'$	15.02^k	$-19.067'$	-286.381
②	$12(14.3 + 7'\text{-}4\frac{3}{8}) = 260.5\square'$	45.58^k	-3.447	-157.109
③	$(3'\text{-}11\frac{3}{8})\left(\frac{12}{2}\right) = 23.69\square'$	4.15^k	$+8.722$	$+36.197$
		64.75^k		$\Sigma M = -407.29'^k$

HAUNCHES :- VOL. $10.5469 \times 18.669' = 196.897\,ft^3$

$Wt = 29.53^k$

$\bar{x} = -21.45$ $M = -633.42'^k$

TOTAL WT = $64.75^k + 29.53^k = 94.28^k$

$M = -407.29 - 633.42 = -1040.71'^k$

$\bar{x} = \underline{\underline{-11.04'}}$

RESISTING MOMENT DUE TO WT OF BUTTRESS

WALL "A" $M_R = 47.52 \times (4.162 + 11.354) = 737.32'^k$

WALL "B" $M_R = 94.28 \times (11.04 + 11.354) = 2111.31'^k$

$2848.63'^k$

WT. OF WATER AT 0.5PMF = 490.6

$5.6' \times 28.6 \times 15 \times 62.4 = 149.91^k \times (14.3 + 11.354) = 3845.79^{'k}$

$\frac{1}{2} \times 28.6 \times 24 \times 62.4 \times 15 = 321.68^k \times (\frac{2}{3} \times 28.6 + 11.354) = 9785.72^{'k}$

TOTAL $M_R = 3845.79 + 9785.72 + 2848.63 = 16480.14^{'k}$

HYDROSTATIC PRESSURE :-

$H = (.5)(29.6)^2(62.4)(15) = 410^k$

$M_{OT} = 410 \times \frac{29.6}{3} = 4045.756^{'k}$

$F.S. = \frac{16480.14}{4045.796} = 4.07$ FACTOR SAFETY OF OVERTURNING @ 0.5MPF AT EL. 461.

$H_u = 574^k$

TOTAL LENGTH OF BUTTRESS AT EL. 461 = 40'

No. OF 1'-0 KEY SAY 6

EA. KEYS WILL CARRY $\frac{574}{3} = 191.3^k$

$V_u = \frac{15 \times 191.3}{(.85)(84)(14)} = 287 psi$

$V_u - V_c = 287 - 110 = 177 < 438$ $\therefore$ OK!

MAX B'R'G FORCE $1.785 \times 6 \times 14 = 149.94^k$

$\Delta H_u = 574 - 149.94 = 424.06^k$

TOTAL D.W. = $149.91 + 321.68 + 94.38 + 47.52 = 613.49^k \times 1.4 = 858.9$

$F_{MAX} = .858.9 > 424.06^k$

$F.S.^* = \underline{\underline{2.03}}$ FACTOR OF SAFETY AGAINST SLIDING AT EL. 461 FOR 0.5MPF.

* SEE NOTE ON SHT 8

BY ________ DATE ________ SUBJECT ________ SHEET 8 OF 10

CHKD. BY ________ DATE ________ JOB No. ROCK RUN DAM.

WT. of WATER AT PMF = 493.9

$8.5 \times 28.6 \times 15 \times .0624 = 227.54 \times (11.3 + 11.354) = 5837.31^{'k}$

$\frac{1}{2} \times 28.6 \times 24 \times 62.4 \times 15 = 321.65^{k} \times (\frac{2}{3} \times 28.6 + 11.354) = 9755.72^{'k}$

TOTAL M_R = 5837.31 + 9755.72 + 2848.63 = $18471.65^{'k}$

TOTAL W_T = 227.54 + 321.68 + 94.38 + 47.52 = 643.6^{k}

HYDROSTATIC PRESSURE:–

$H = (.5)(32.5)^2(62.4)(15) = 494.33^{k}$

$M_{OT} = 494.33 \times \frac{32.5}{3} = 5355.19^{'k}$

$F.S. = \frac{18471.65}{5355.19} = 3.45$ FACTOR OF SAFETY AGAINST OVERTURNING @ EL. 461 FOR PMF.

$H_U = 692.06^{k}$

$V_U = \frac{692.06}{3} = 230.7^{k}$ / KEY

$V_U = 346 psi$

$V_U - V_C = 346 - 110 = 236 psi < 438$ ∴ OK!

$\Delta H_U = 692.06 - 149.94 = 542.12$

$F_{MAX} = 643.6^{k} \times 1.4 = 901.04$

$F.S. = \frac{901.04}{542.12} = 1.66$ FACTOR OF SAFETY AGAINST SLIDING AT EL. 461 FOR PMF.

NOTE:– THIS FACTOR OF SAFETY COMPUTED BASE ON ULTIMATE STRENGTH THEORY WITH L.F. = 1.4.

BY DC DATE 9/22/78 SUBJECT ________ SHEET 9 OF 10

CHKD. BY [illegible] DATE 9/25/78 JOB No. ROCK RUN DAM.

IN VIEWING THE COMPUTATION, THE WT. OF WATER PLAYS THE MAJOR PART IN COUNTERBALANCING OVERTURNING AND SLIDING. THUS, JUST USING THE WT. OF WATER, NEGLECTING WT OF STRUCTURE, MINIMUM FACTOR OF SAFETY FOR THE STRUCTURE CAN BE DETERMINED. FURTHER, PMF CONDITION YIELDS THE LOWEST FACTOR OF SAFETY. IF THE STRUCTURE DOES NOT FAIL UNDER PMF CONDITION, IT CAN NOT FAIL AT 0.5PMF.

AT ELEV. 449. FOR PMF

WT. OF WATER: $3 \times 14.3 \times 8.5 \times 15 \times .0624 = 341.3^k \times (1.5 \times 14.3 + 18) = 13464.77^{k}$

$3 \times 14.3 \times 36 \times \frac{15}{2} \times .0624 = 722.78^k \times (\frac{2}{3} \times 14.3 \times 36 + 18) = 33681.55^{'k}$

$M_R = 13464.77 + 33681.55 = \underline{\underline{47146.32}}^k$

$WT\ OF\ WATER = 341.3 + 722.78 = 1064.08 \times 1.4 = \underline{\underline{1489.7^k}}$

$F_{MAX} = 1489.7^k$

HYDROSTATIC PRESSURE :-

$H = \frac{44.5^2}{2} \times .0624 \times 15 = 926.76^k$

$H_U = 1297.5^k$

$H_{D/S\ MAX} = 6 \times 16 \times 1.785 = 171.36^k$

OVERTURNING. $M_0 = 926.76 \times \frac{44.5}{3} = 14191.94^{'k}$

$F.S._{MIN} = \frac{33681.55}{14191.94} = \underline{\underline{2.37}}$

SLIDING $\Delta H = 1297.5 - 171.4 = 1126.1^k$

$F.S._{MIN} = \frac{1459.7}{1126.1} = \underline{\underline{1.32}}$

MINIMUM FACTORS OF SAFETY AGAINST OVERTURNING AND SLIDING

BY E= DATE 9/22/78 SUBJECT

CHKD. BY [illegible] DATE 9/25/78

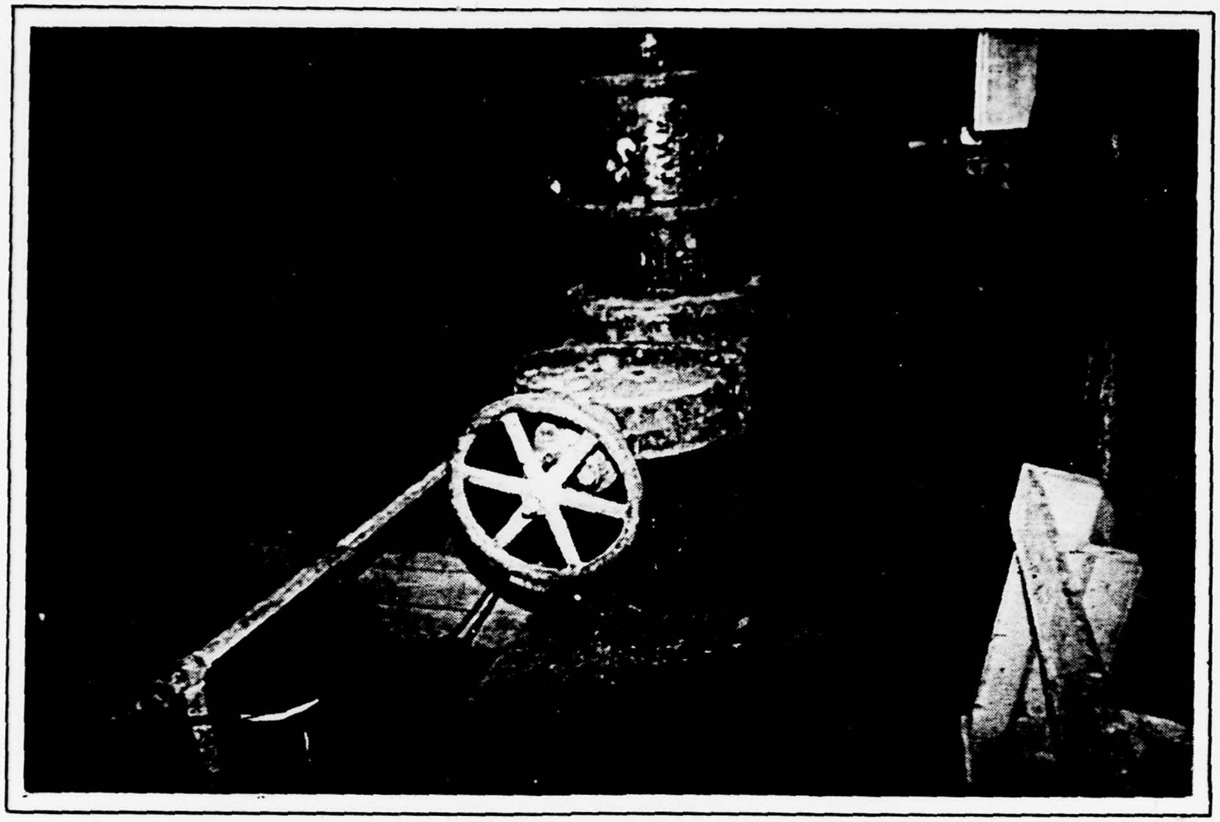
SHEET 10 OF 10

JOB No. ROCK RUN DAM.

CONCLUSION : THIS SET OF CALCULATIONS INDICATE THE DAM IS SAFE UNDER BOTH 0.5 PMF AND PMF CONDITIONS.

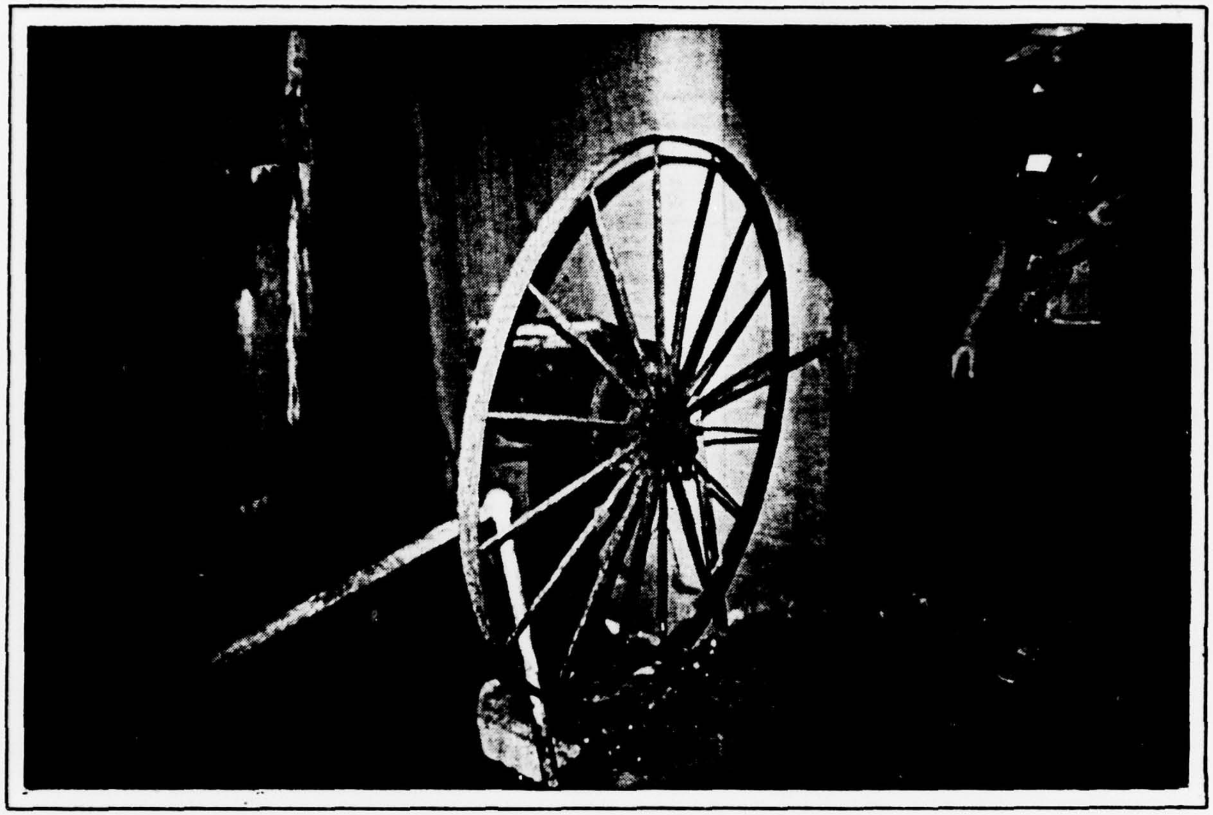
APPENDIX

D



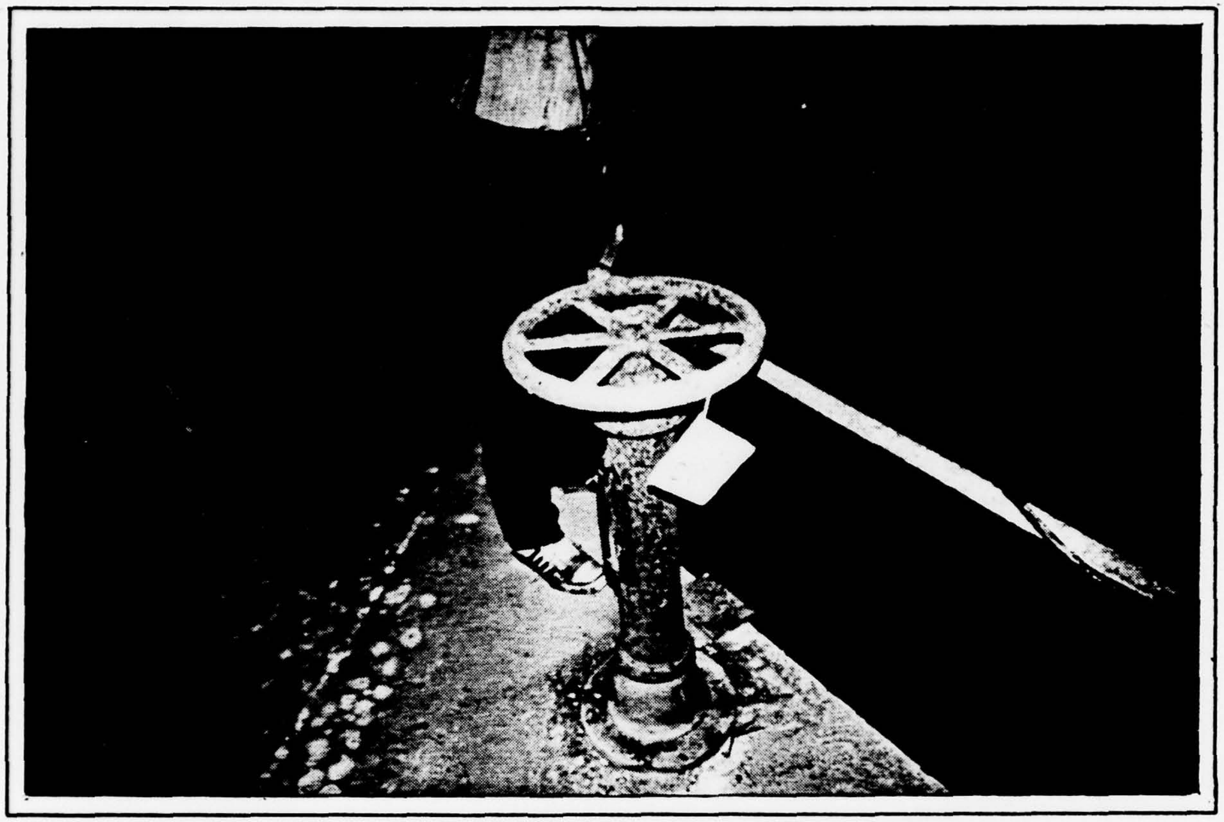
UPPER INTAKE VALVE WITH FLOW GAGE
ATTACHED TO WALL. THIS IS A NEW
VALVE WHICH IS ALWAYS OPEN.

PHOTOGRAPH NO. 1



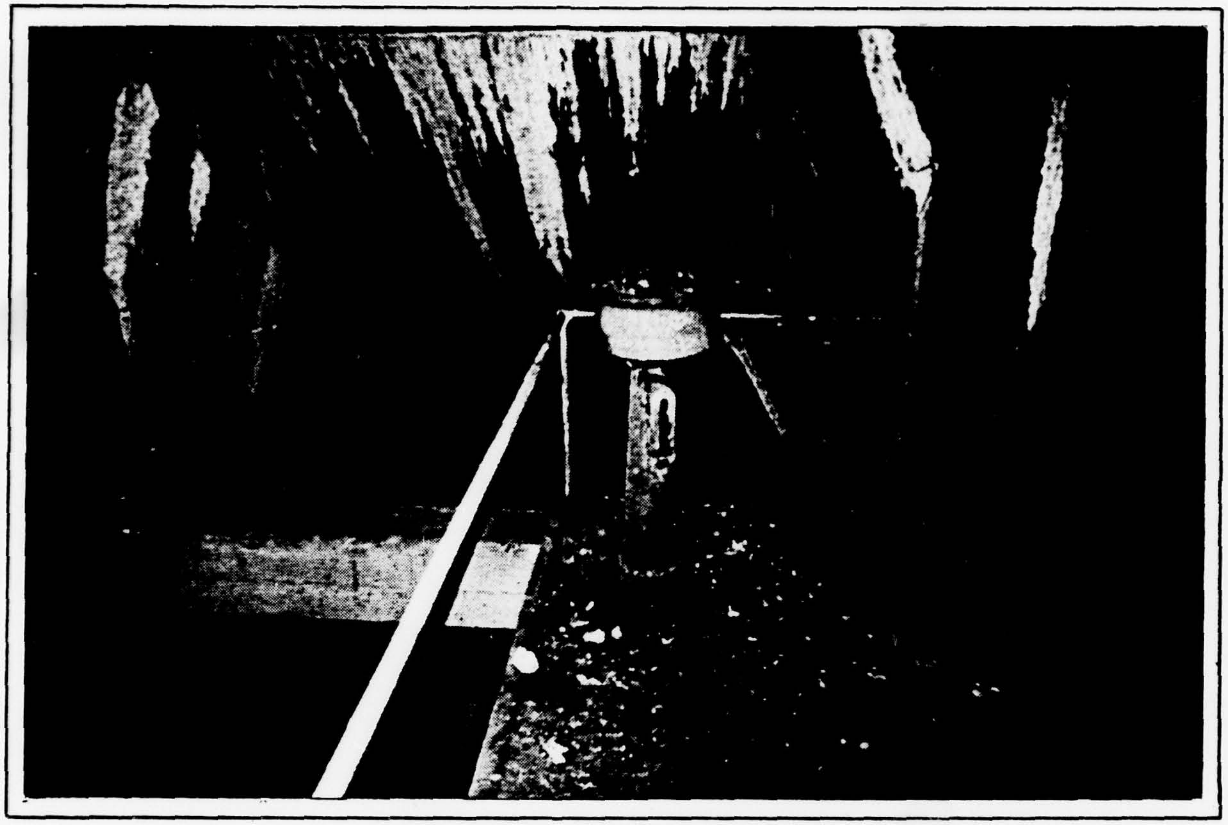
ABANDONED POND DRAIN VALVE THAT LEAKS AND IS LOCKED (FROM RUST) IN THE CLOSED POSITION. INTAKE PIPE IS REPORTEDLY FILLED WITH SEDIMENT.

PHOTOGRAPH NO. 2



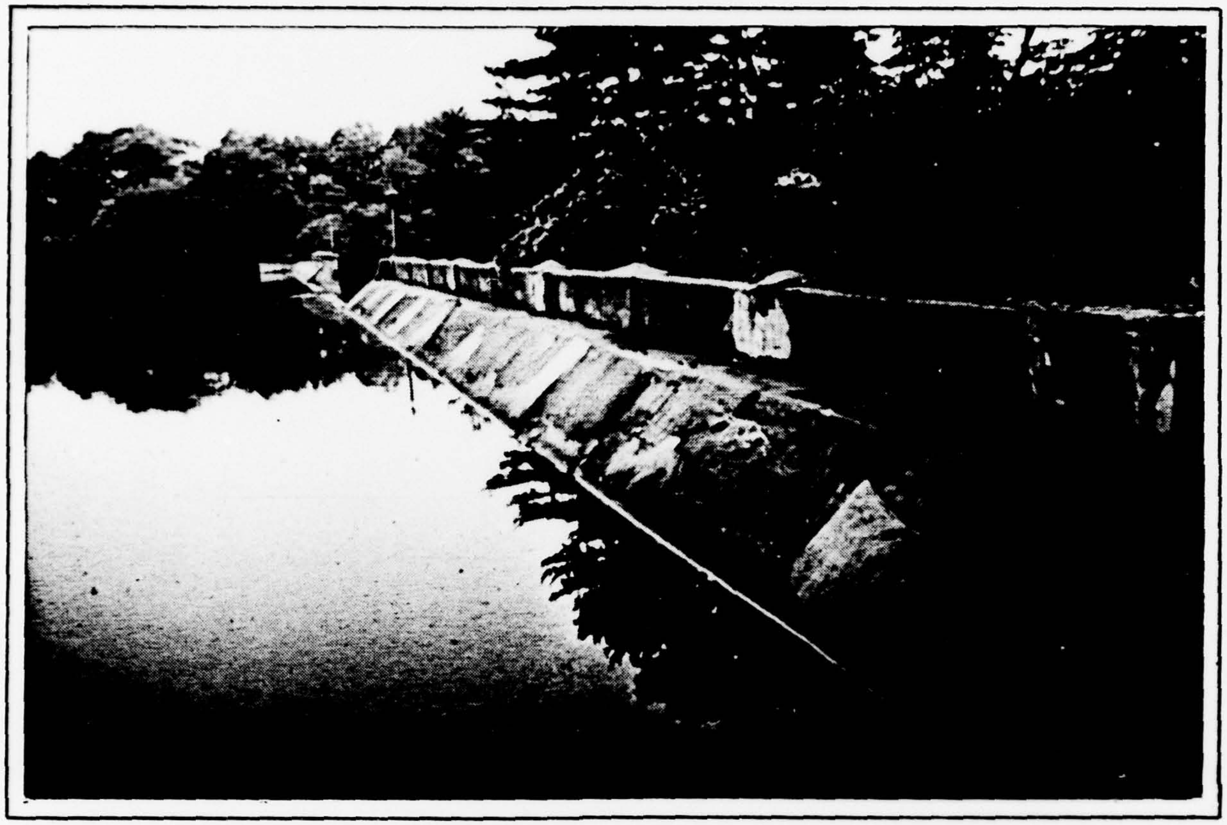
ABANDONED VALVE FOR MIDDLE INTAKE PIPE.

PHOTOGRAPH NO. 3



ABANDONED DIVERSION PIPE RUSTED
IN CLOSED POSITION.

PHOTOGRAPH NO. 4

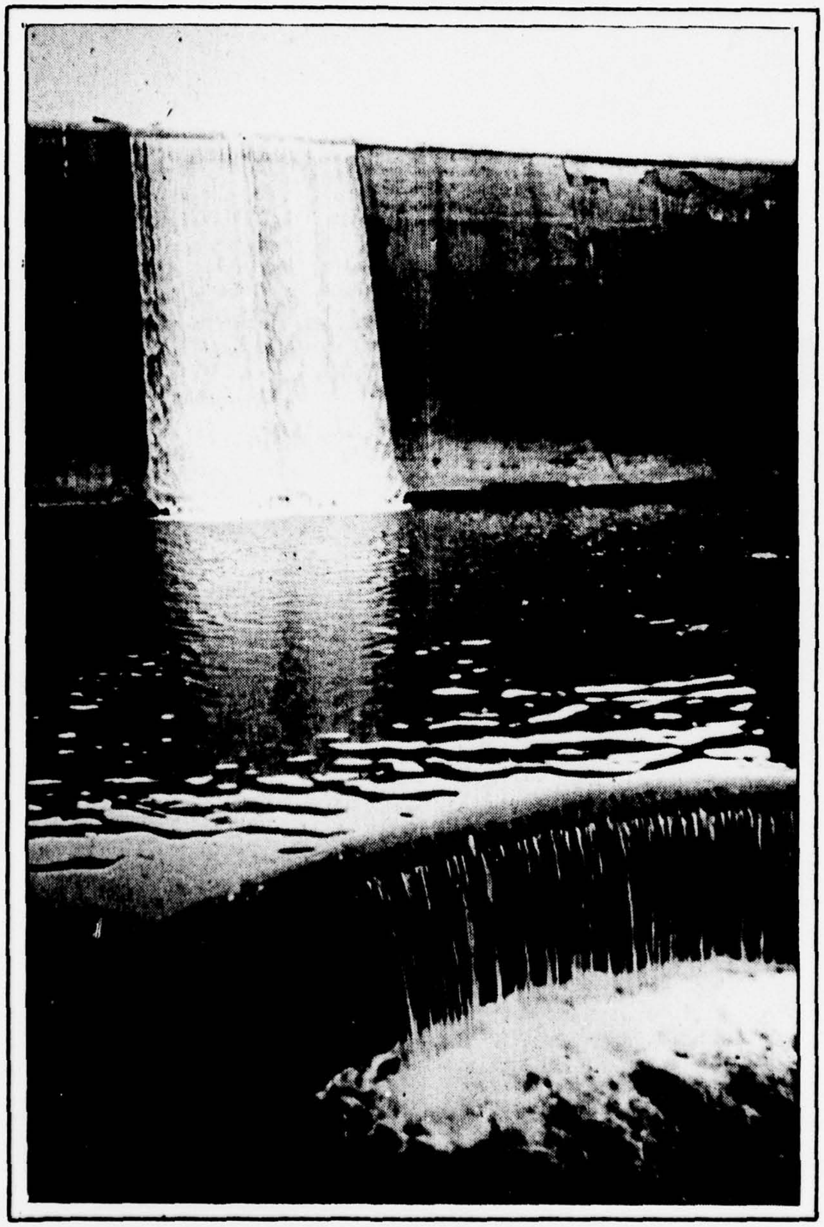


UPSTREAM FACE OF DAM. NOTE PATCHED AREAS WHERE REPAIR WORK WAS PERFORMED.

PHOTOGRAPH NO. 5

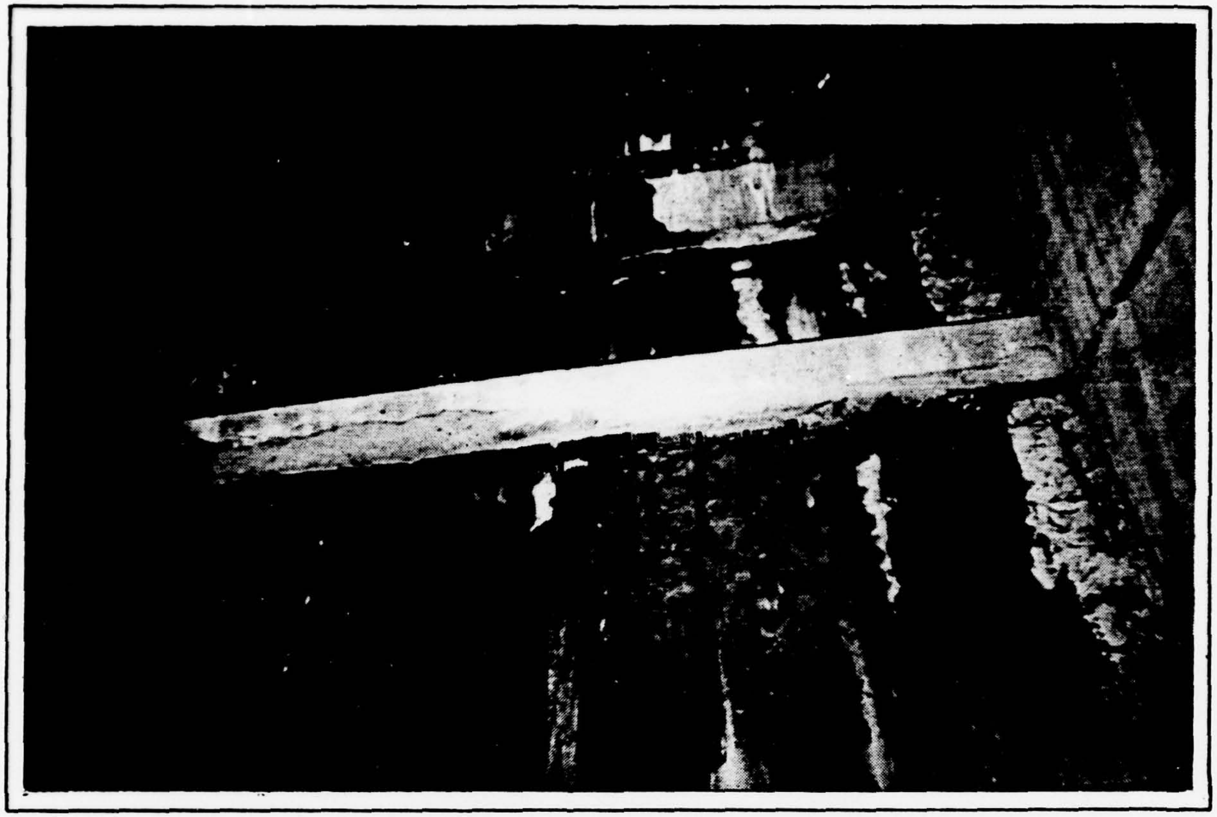
VIEW OF DOWNSTREAM BUTTRESSES AND FACIAL WALLS BETWEEN BUTTRESSES. FACIAL WALLS ARE NON-LOAD BEARING WALLS.

PHOTOGRAPH NO. 6



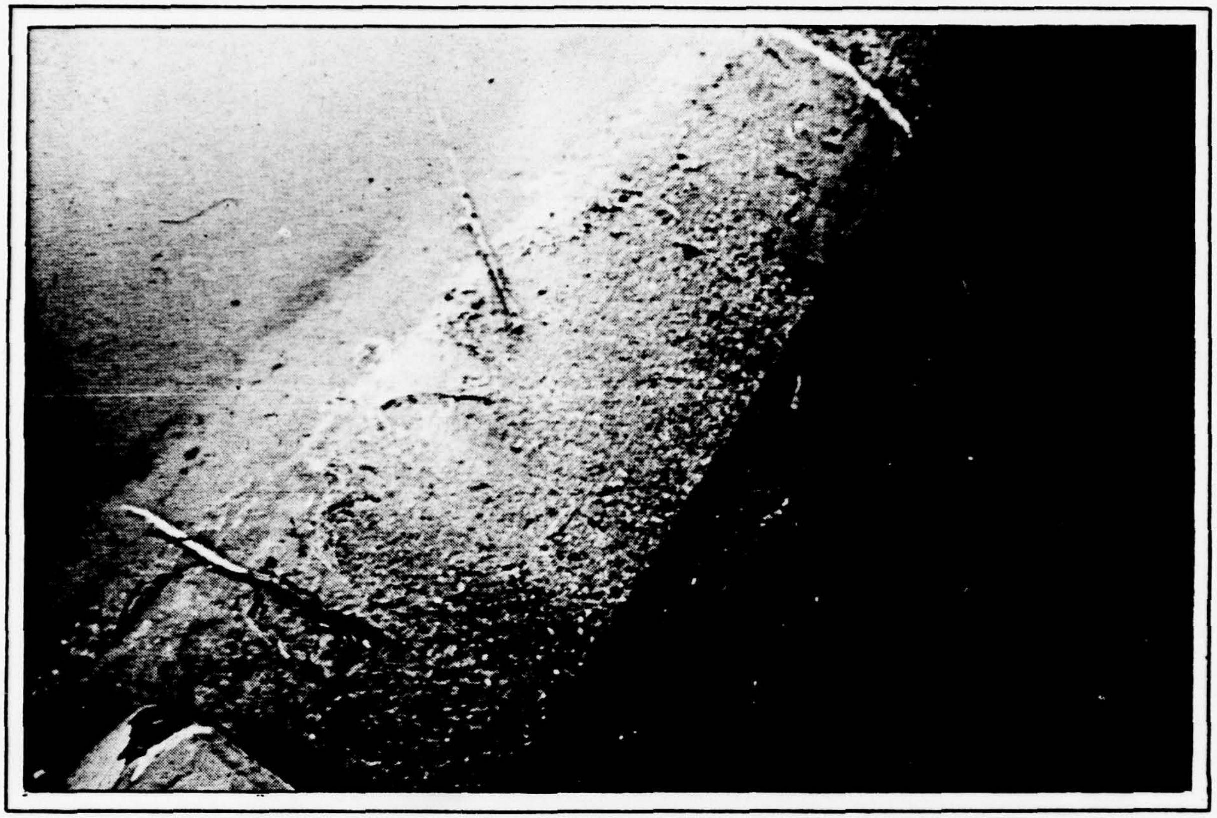
DOWNSTREAM STILLING BASIN
AND OVERFLOW WEIR.

PHOTOGRAPH NO. 7



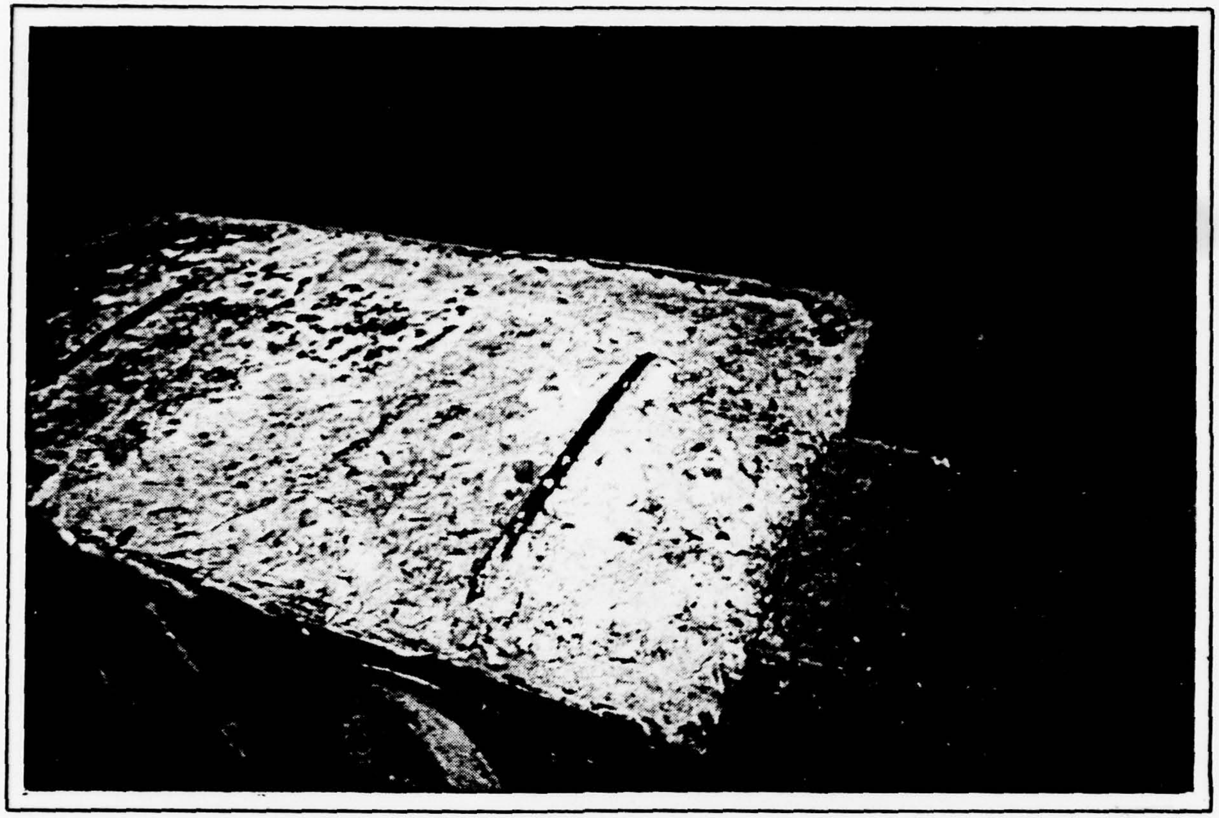
VIEW FROM INSIDE OF DAM LOOKING DOWNSTREAM TOWARDS FACIAL WALLS AND BUTTRESS TIE BEAM. NOTE WALL DETERIORATION AND MOVEMENT. MANY OF THE TIE BEAMS ARE ALSO DETERIORATED.

PHOTOGRAPH NO. 8



CLOSEUP VIEW OF SPILLWAY CREST.
NOTE SEPARATION (1 to 2 INCHES)
BETWEEN CAP AND CHUTE.

PHOTOGRAPH NO. 9

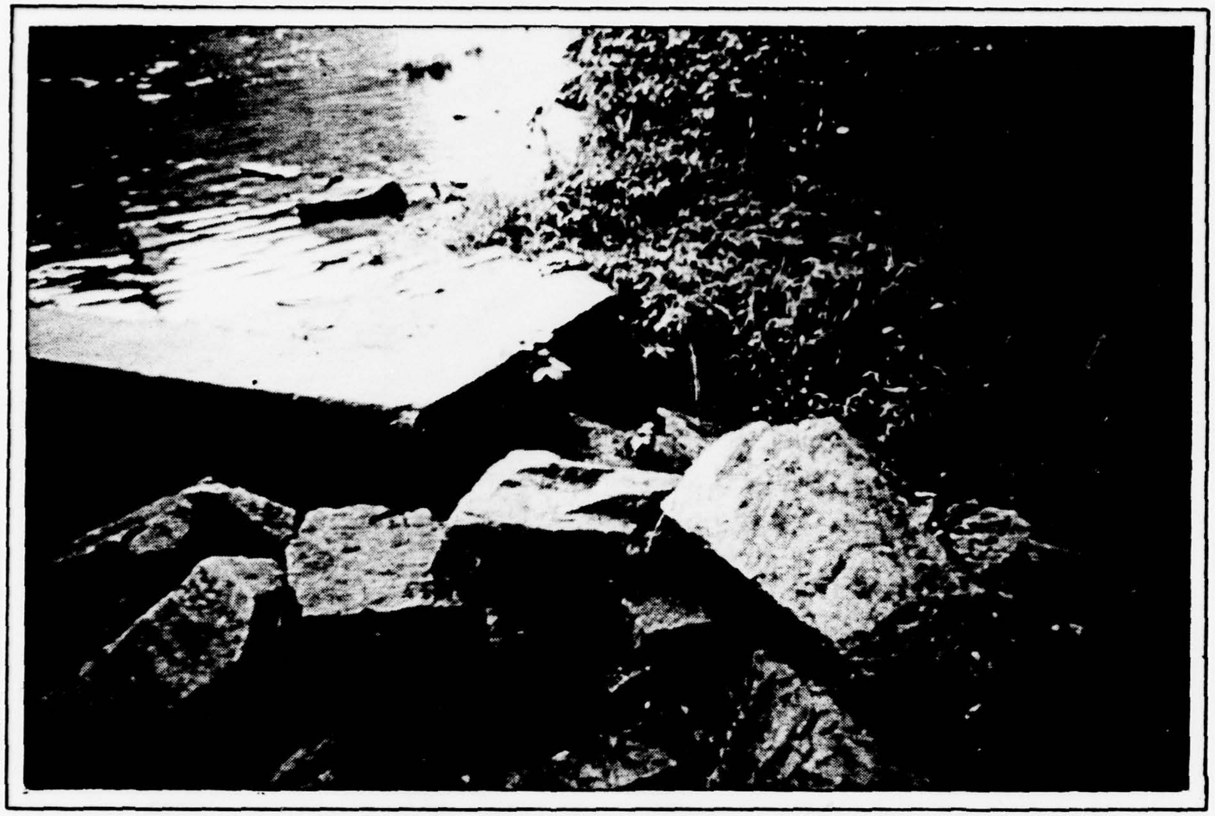


TOP OF A TYPICAL BUTTRESS. NOTE SPALLING, DETERIORATION AND EXPOSED REINFORCING BARS.

PHOTOGRAPH NO. 10

SEEPAGE WAS NOTED THROUGH
JOINTS OF STILLING BASIN.
NOTE ABANDONED STILLING
BASIN DRAIN PIPE.

PHOTOGRAPH NO. 11



SEEPAGE WAS NOTED THROUGH ABUTMENTS OF STILLING BASIN WALLS.

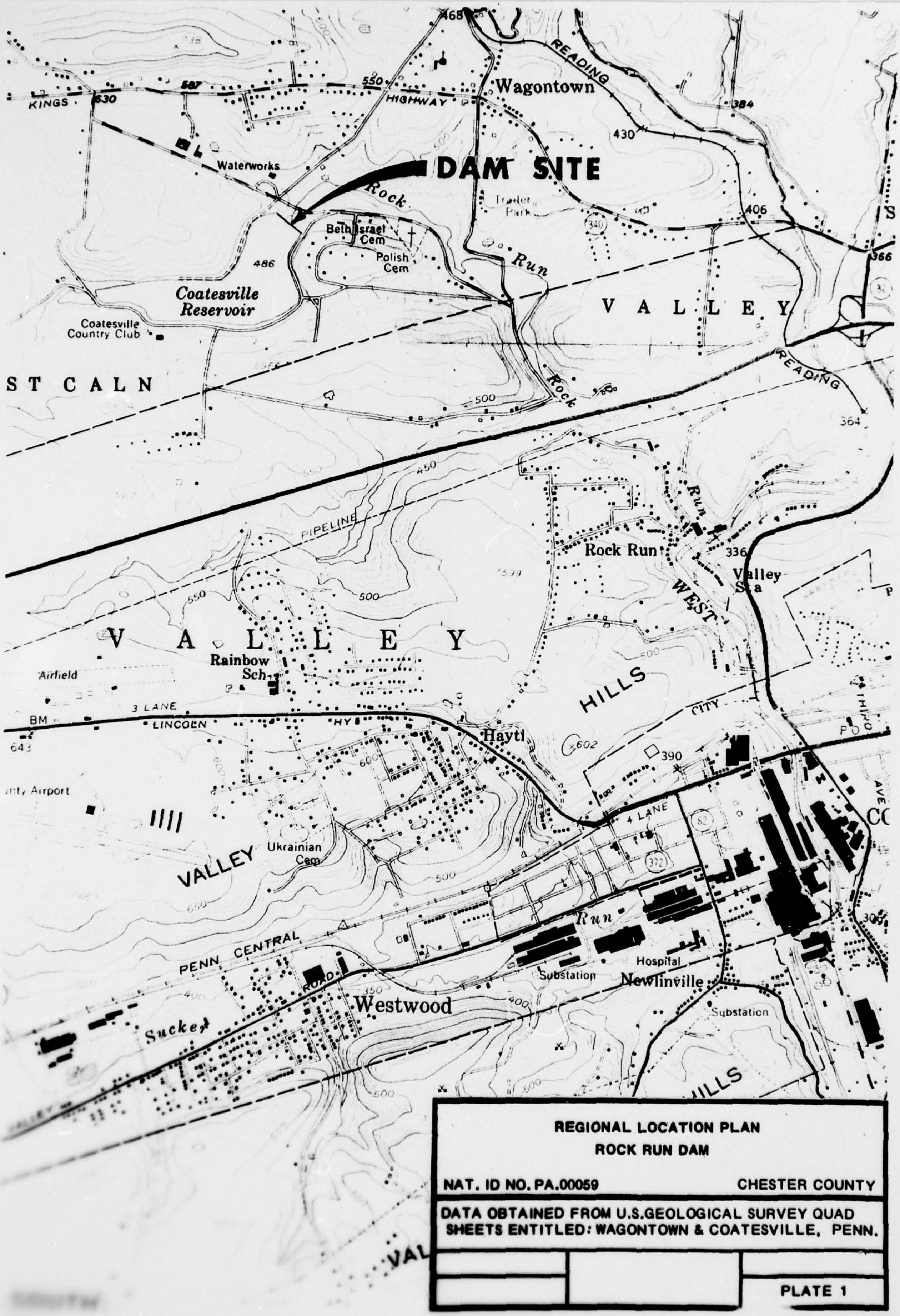
PHOTOGRAPH NO. 12

SEEPAGE AREA LOCATED DOWNSTREAM OF DAM TO THE RIGHT OF THE STILLING BASIN. CATTAILS DELINEATE AREA.

PHOTOGRAPH NO. 13

APPENDIX

E

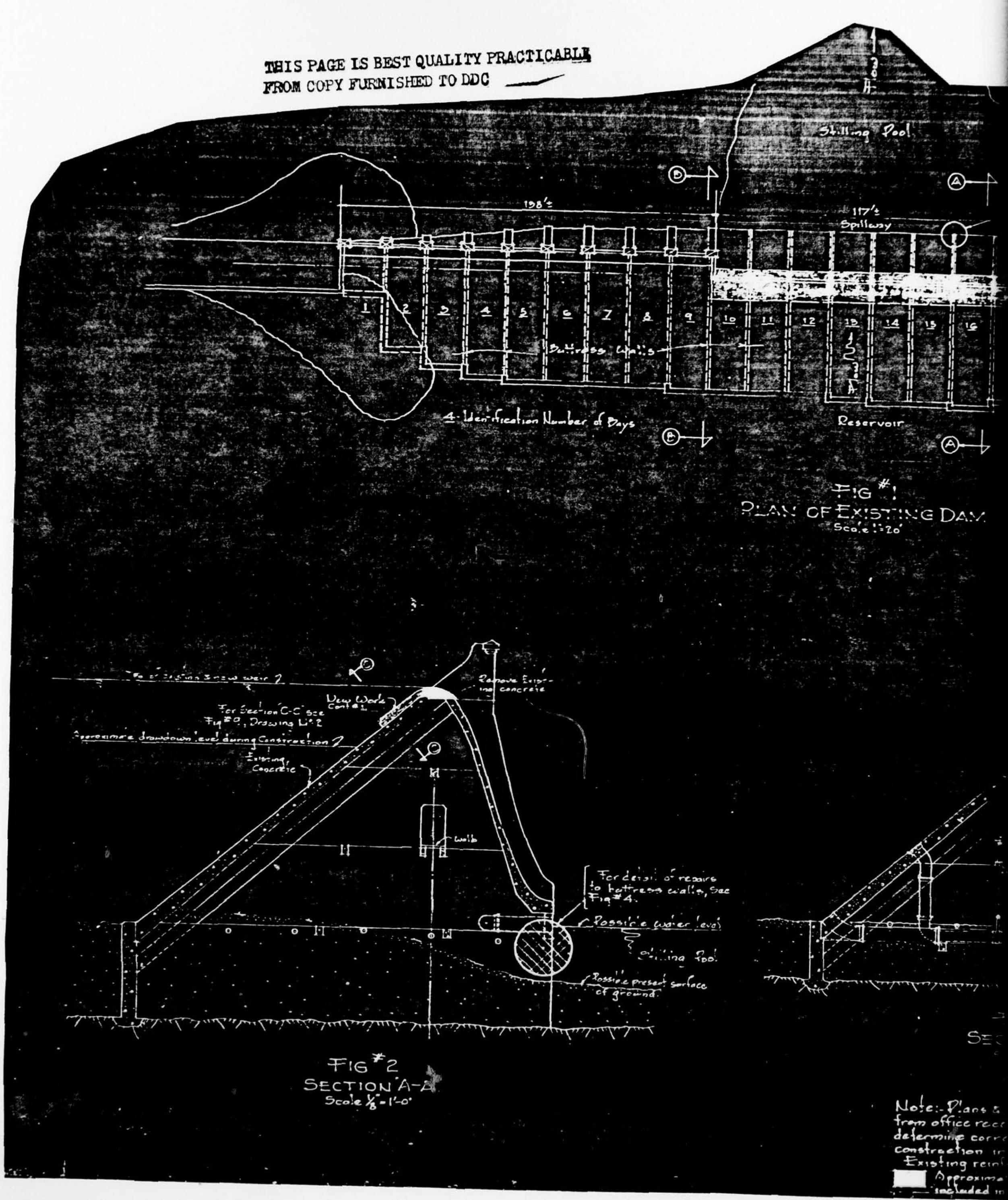


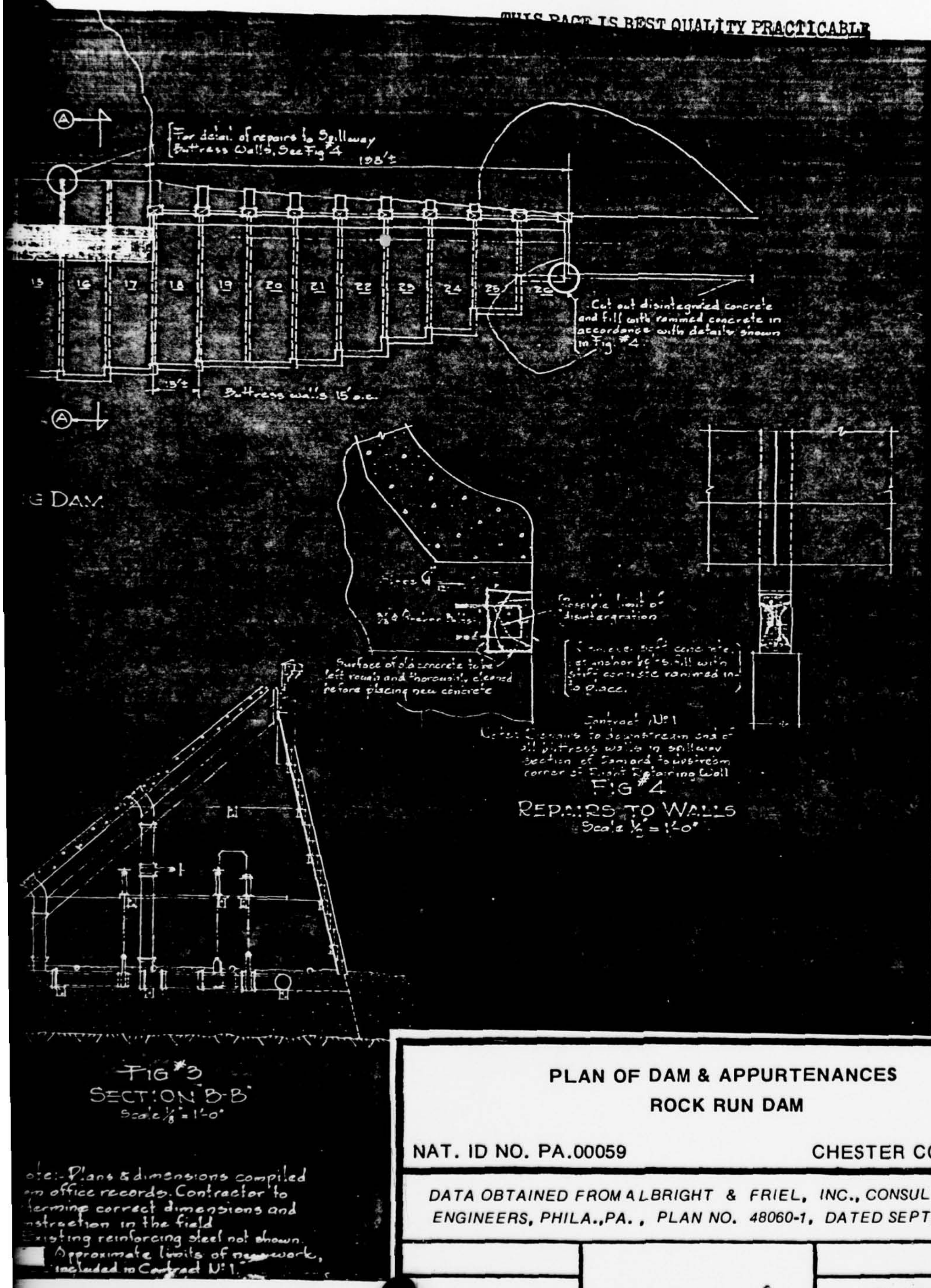
REGIONAL LOCATION PLAN
ROCK RUN DAM

NAT. ID NO. PA.00059	CHESTER COUNTY

DATA OBTAINED FROM U.S.GEOLOGICAL SURVEY QUAD SHEETS ENTITLED: WAGONTOWN & COATESVILLE, PENN.

PLATE 1


Stilling Pool
198'±
117'±
Spillway
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Buttress Walls
4 - Identification Number of Bays
Reservoir
FIG # 1
PLAN OF EXISTING DAM
Scale 1"=20'
Remove Existing concrete
New Work
For Section C-C see Fig # 9, Drawing N° 2
Approximate drawdown level during Construction
Existing Concrete
Walk
For detail of repairs to buttress walls, See Fig # 4.
Possible water level
Stilling Pool
Possible present surface of ground
FIG # 2
SECTION A-A
Scale 1/8"=1'-0"



PLAN OF DAM & APPURTENANCES

ROCK RUN DAM

NAT. ID NO. PA.00059 CHESTER COUNTY

DATA OBTAINED FROM ALBRIGHT & FRIEL, INC., CONSULTING ENGINEERS, PHILA., PA., PLAN NO. 48060-1, DATED SEPT., 1949

2

PLATE 2

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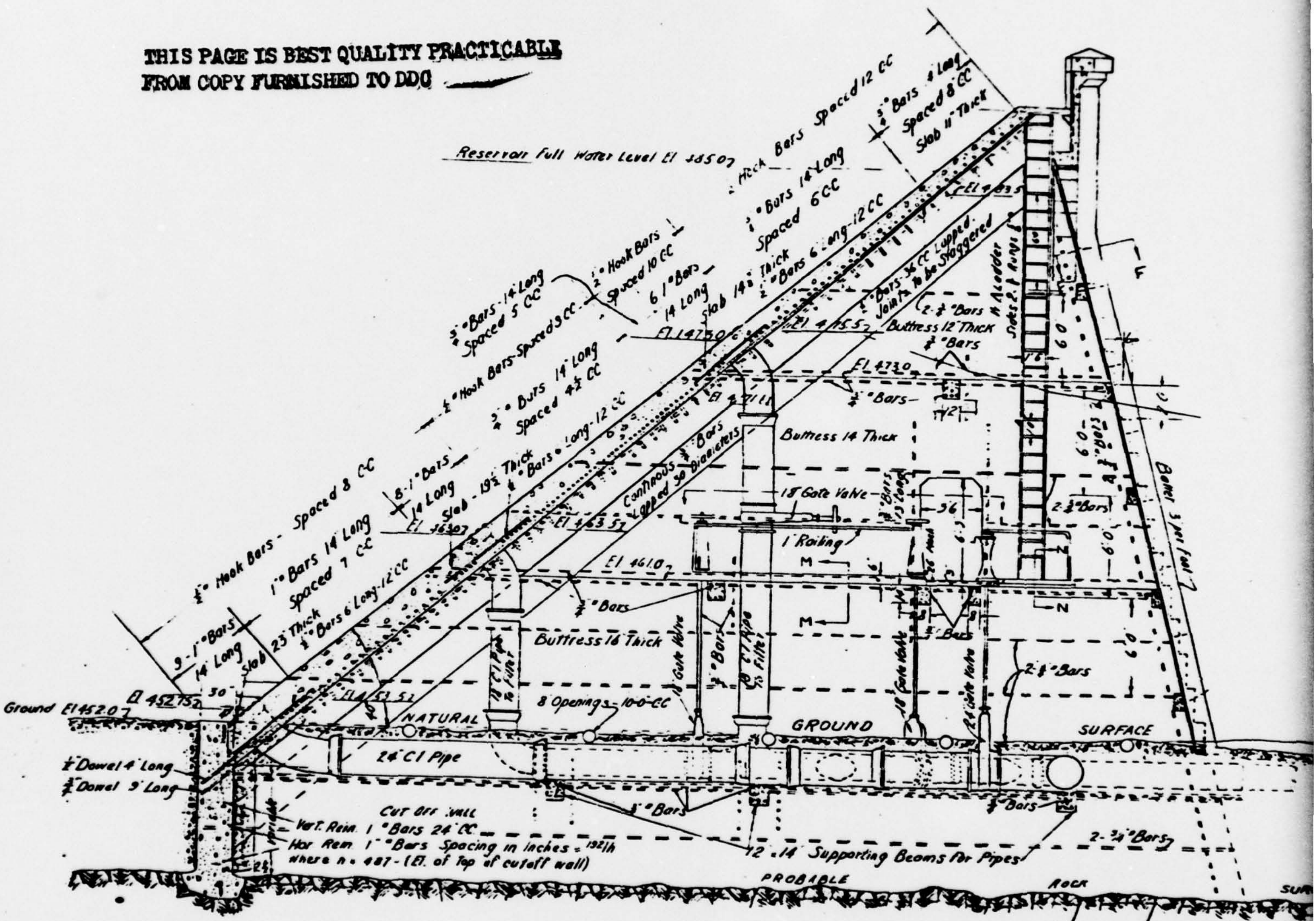
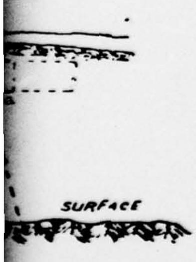


DETAIL SECTION B-B
SCALE ¼" = 1'-0"



SECTION THROUGH INTAKE PIPES
ROCK RUN DAM

NAT. ID NO.PA.00059 CHESTER COUNTY

DATA OBTAINED FROM 1913 WATER-WORKS BOND ISSUE
COATESVILLE, PA.

	2	PLATE 3

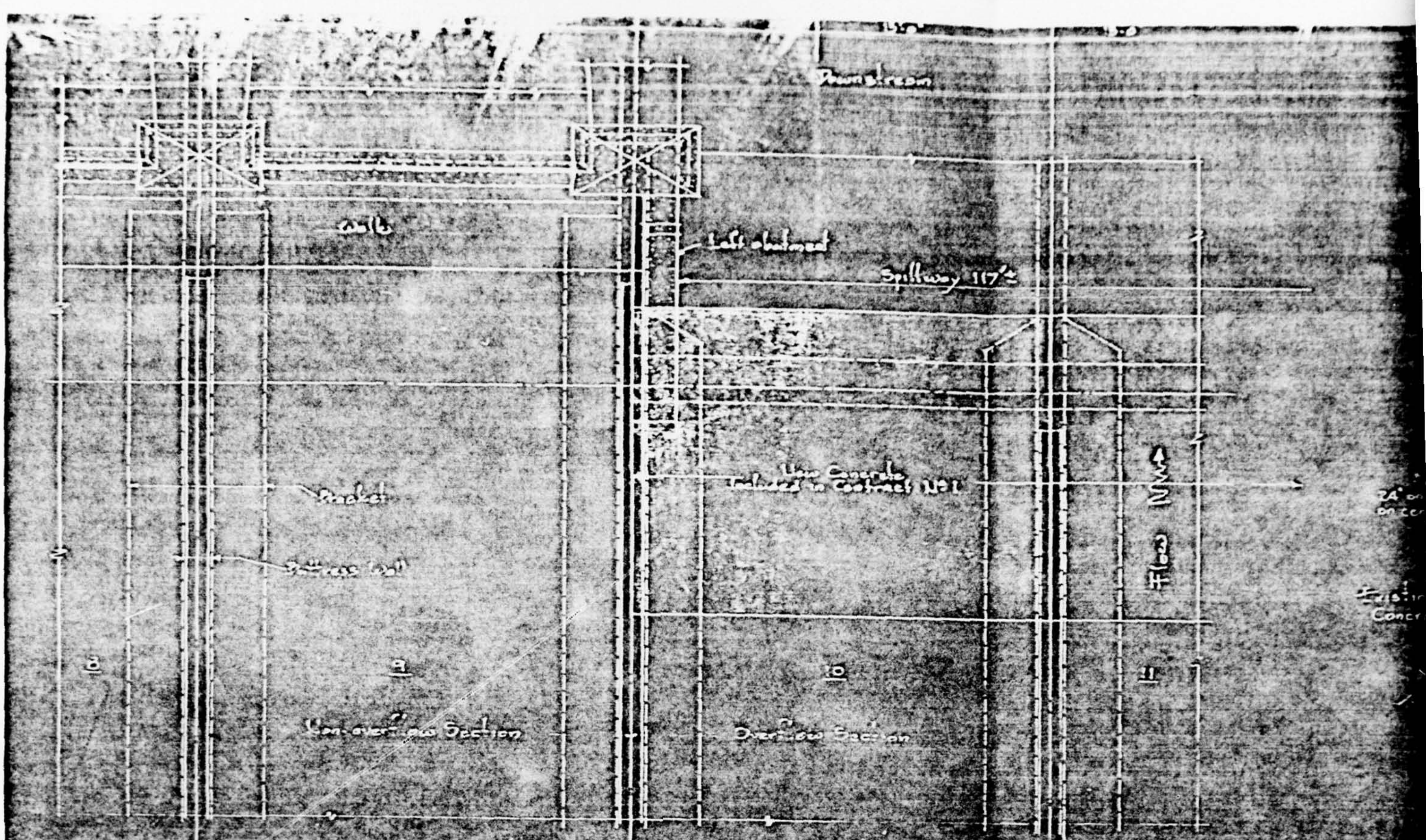


FIG #5
PART PLAN AT LEFT ABUTMENT

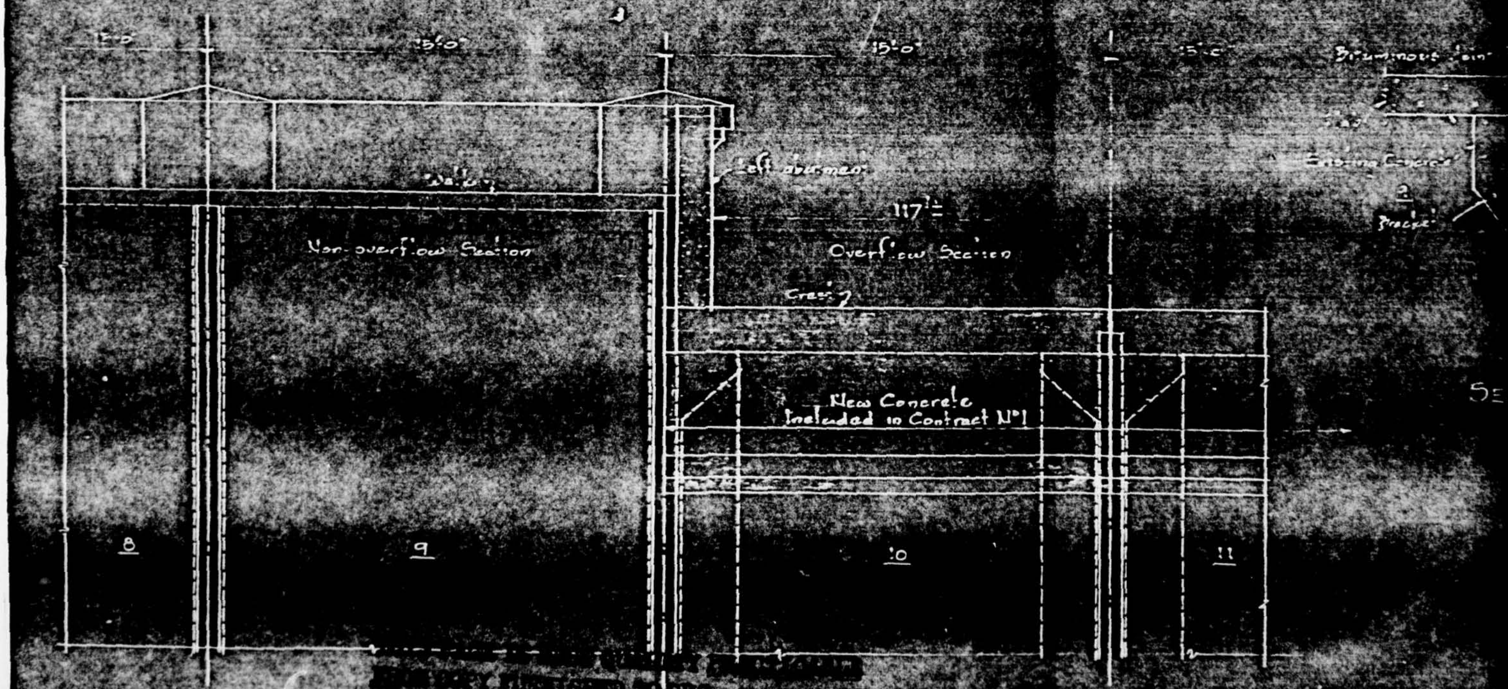
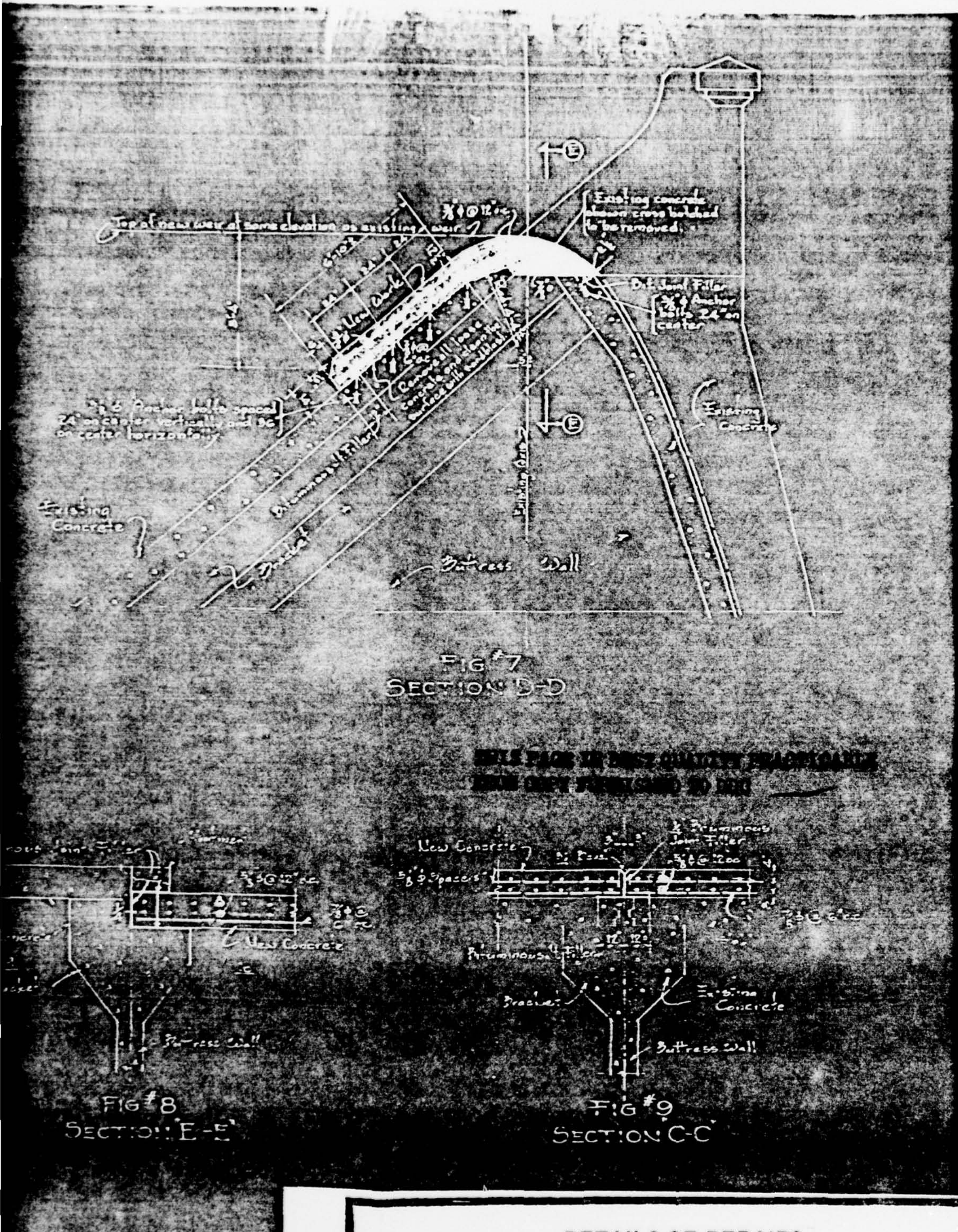


FIG #6
PART ELEVATION AT LEFT ABUTMENT

Note:- Plans & dimensions com
from office records. Contractor
determine correct dimensions



DETAILS OF REPAIRS
ROCK RUN DAM

NAT. ID NO. PA.00059 CHESTER COUNTY

DATA OBTAINED FROM ALBRIGHT & FRIEL, INC., CONSULTING ENGINEERS, PHILA.,PA., PLAN NO. 48060-2, DATED SEPT.,1949

PLATE 4

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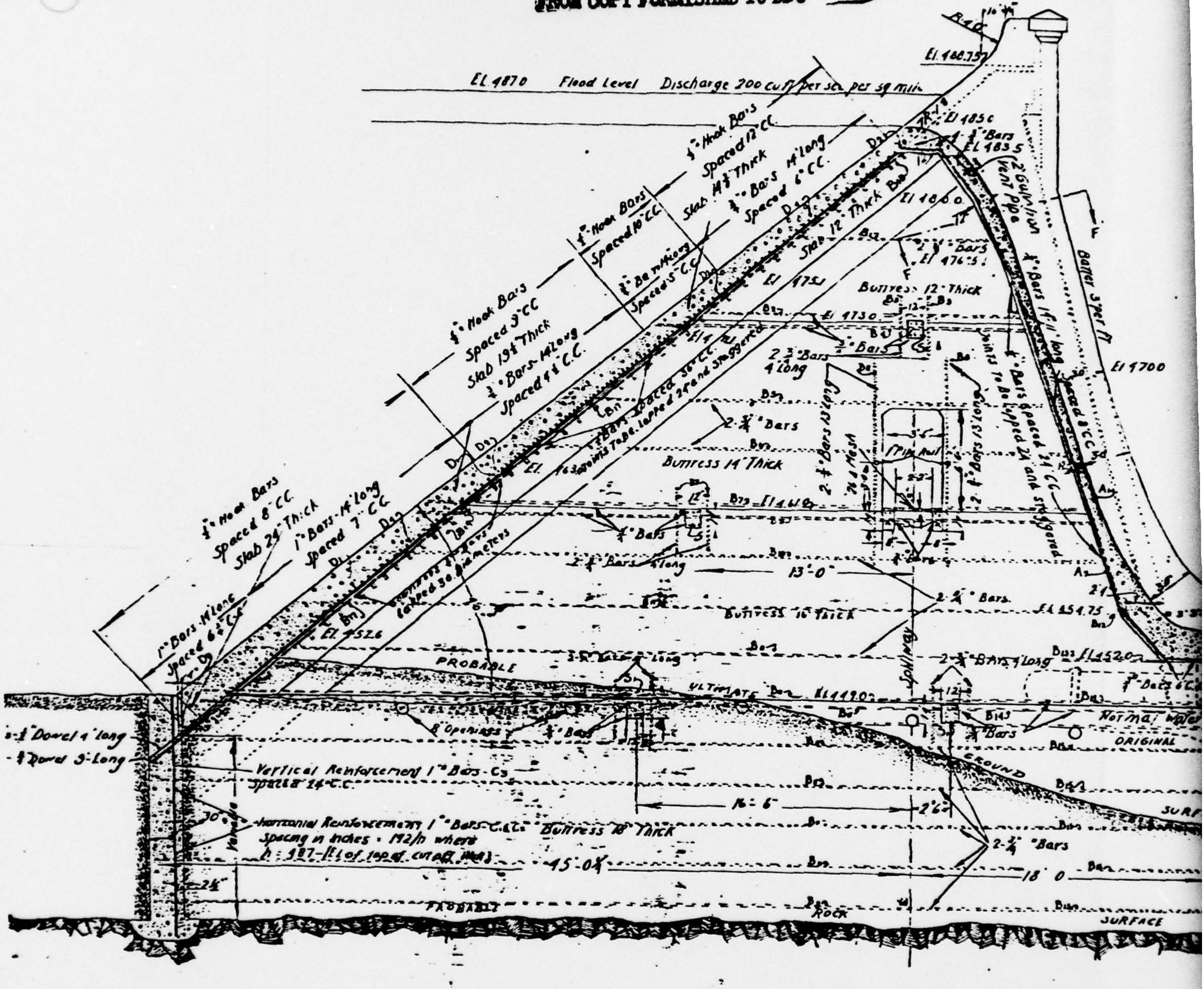
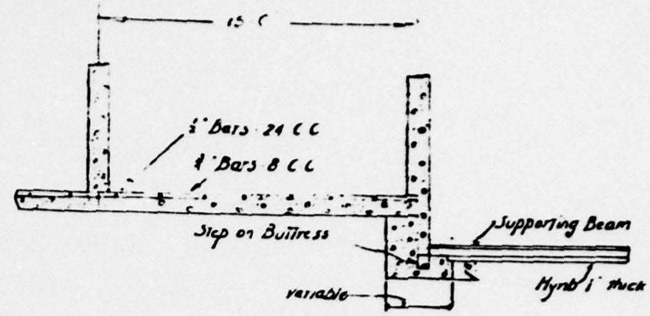
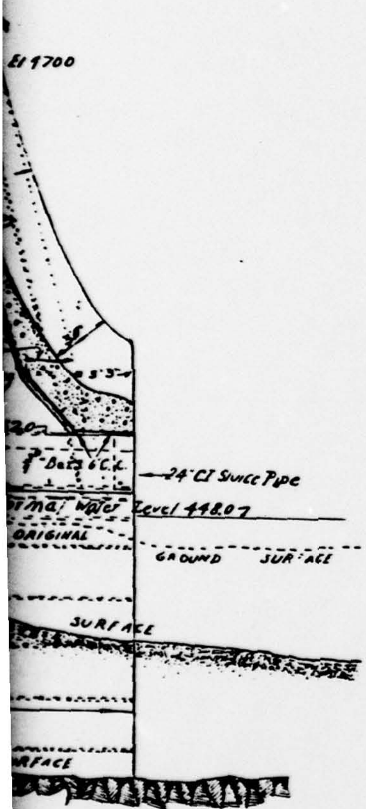
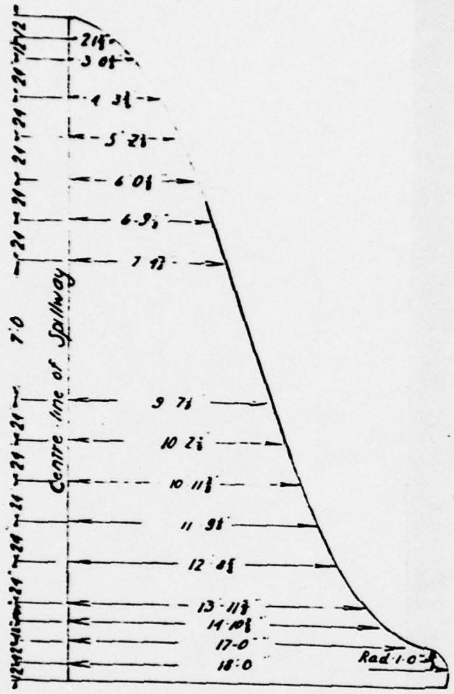


SPILLWAY SECTION A-A

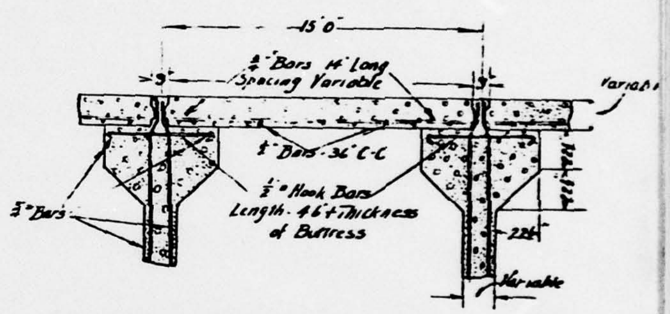


SECTION F'F'
AT JUNCTION OF SPILLWAY
AND
MAIN SECTION OF DAM

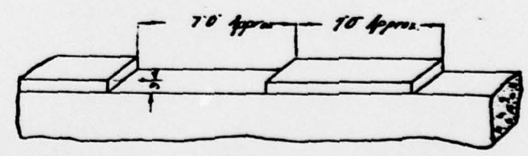




OFFSETS FOR SPILLWAY CURVE



GENERAL SECTION THROUGH DECK



BUTTRESS FIELD JOINT

TYPICAL SECTION OF SPILLWAY

ROCK RUN DAM

NAT. ID NO. PA.00059 CHESTER COUNTY

DATA OBTAINED FROM 1913 WATER-WORKS BOND ISSUE
COATESVILLE, PA.

PLATE 5

APPENDIX

F

SITE GEOLOGY
ROCK RUN DAM

The Rock Run Dam is located in the Piedmont Uplands section of the Piedmont Physiographic Province. The bedrock at the dam site is reported to consist of the Pre-cambrian Baltimore Gneiss (see Plate F-1). In the vicinity of the dam, the Baltimore Gneiss is bounded to the north and to the south by lower Paleozoic gabbro. This formation is cut by the lower Paleozoic pegmatites to the south of the dam (Poth, 1968). The Baltimore Gneiss is reported to have been intensely deformed and metamorphosed, with fold axes, foliations, and late stage pegmatite intrusives generally having an east-northeast strike, and having a variable dip (Poth, 1968). Although no joint data are available for the dam site, joints within the Baltimore Gneiss are reported to be numerous, closely spaced, locally filled with epidote and chlorite, and are variable in orientation (USGS Map I-514A, 1967). Two lower Paleozoic faults have been reported just north of the dam, striking approximately east-west.

Very limited Pleistocene deposits have been reported in the region, consisting of occasional thin deposits of glacial outwash (Leverett, 1937). These deposits are assumed to have been removed from the vicinity of the dam structure during construction.

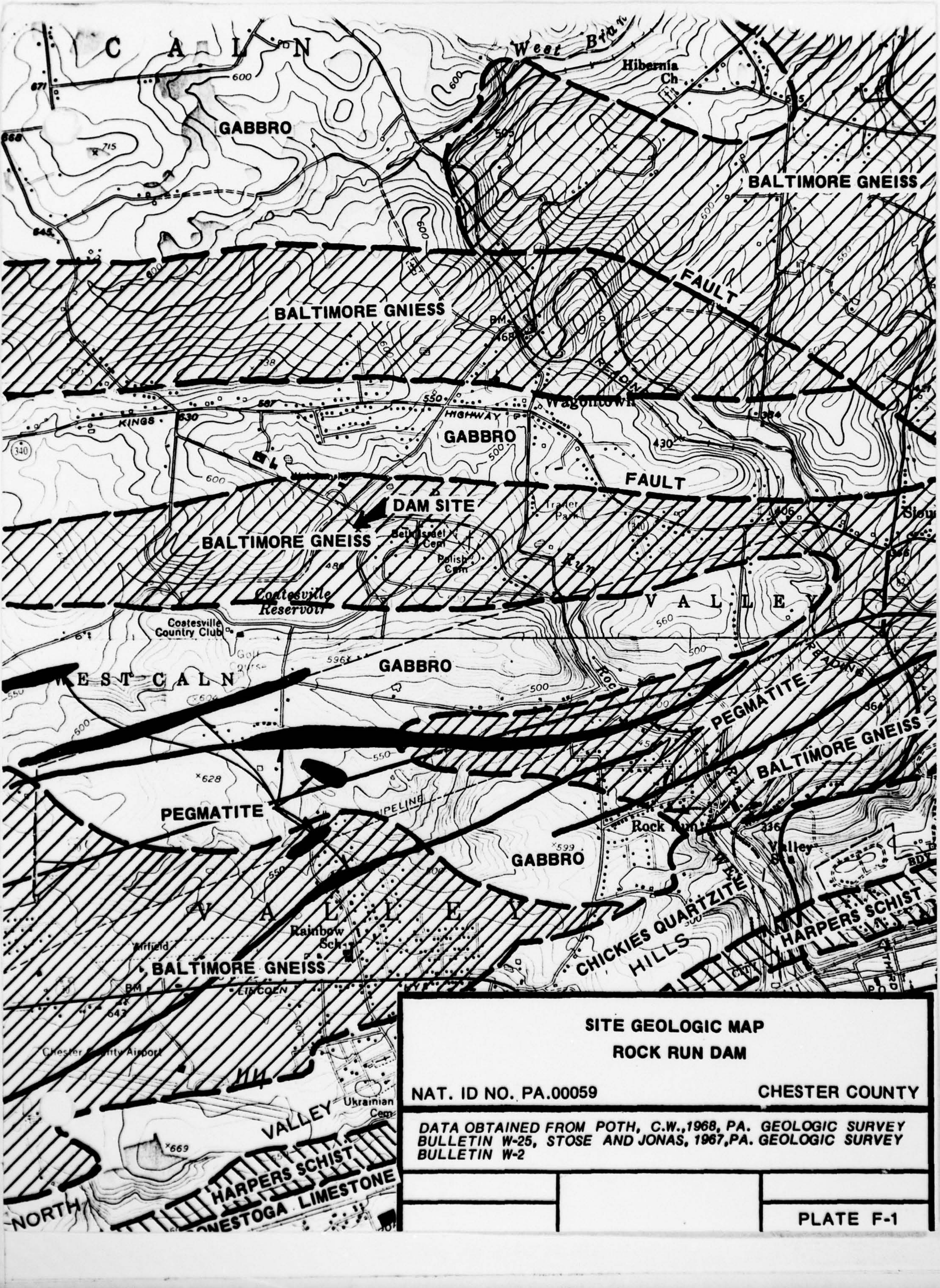
Downstream seepage should not be a serious problem unless the reported east-west trending faults north of the dam have any associated fracture systems that are zones of groundwater transport beneath the dam structure, due to the dam being constructed at a non-parallel angle to these fault systems.

References:

1. *Poth, C.W., 1968, Hydrology of the Metamorphic and Igneous Rocks of Central Chester County, Pennsylvania: Pa. Geol. Survey, 4th series, 84 p.*

2. *Stose, G.W. and Jonas, A.I., 1967, Geologic Map of Southeastern Pennsylvania: Pa. Geol. Survey, 4th series, 84 p.*

3. *USGS, 1967, Engineering Geology of the Northeast Corridor Washington, D.C., to Boston, Massachusetts: Bedrock Geology: USGS Miscellaneous Geologic Investigations Map I-514-A, 1:250,000.*



SITE GEOLOGIC MAP

ROCK RUN DAM

NAT. ID NO. PA.00059 CHESTER COUNTY

DATA OBTAINED FROM POTH, C.W.,1968, PA. GEOLOGIC SURVEY BULLETIN W-25, STOSE AND JONAS, 1967, PA. GEOLOGIC SURVEY BULLETIN W-2

PLATE F-1